IN SITU GROUND EVALUATION BY DEEP SOIL MIXING

A thesis submitted in partial fulfilment of the requirements for the Degree

of Master of Science in Engineering Geology

at the University of Canterbury

by

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2017

Abstract

Between 2010 and 2011, Canterbury experienced a series of four large earthquake events with associated aftershocks which caused widespread damage to residential and commercial infrastructure. Fine grained and uncompacted alluvial soils, typical to the Canterbury outwash plains, were exposed to high peak ground acceleration (PGA) during these events. This rapid increase in PGA induced cyclic strain softening and liquefaction in the saturated, near surface alluvial soils.

Extensive research into understanding the response of soils in Canterbury to dynamic loading has since occurred. The Earthquake Commission (EQC), the Ministry of Business and Employment (MBIE), and the Christchurch City Council (CCC) have quantified the potential hazards associated with future seismic events. Theses bodies have tested numerous ground improvement design methods, and subsequently are at the forefront of the Canterbury recovery and rebuild process.

Deep Soil Mixing (DSM) has been proven as a viable ground improvement foundation method used to enhance in situ soils by increasing stiffness and positively altering in situ soil characteristics. However, current industry practice for confirming the effectiveness of the DSM method involves specific laboratory and absolute soil test methods associated with the mixed column element itself. Currently, the response of the soil around the columns to DSM installation is poorly understood. This research aims to understand and quantify the effects of DSM columns on near surface alluvial soils between the DSM columns though the implementation of standardised empirical soil test methods. These soil strength properties and ground improvement changes have been investigated using shear wave velocity (V_s), soil behaviour and density response methods.

The results of the three different empirical tests indicated a consistent improvement within the ground around the DSM columns in sandier soils. By contrast, cohesive silty soils portrayed less of a consistent response to DSM, although still recorded increases. Generally, within the tests completed 50 mm from the column edge, the soil response indicated a deterioration to DSM. This is likely to be a result of the destruction of the soil fabric as the stress and strain of DSM is applied to the un-mixed in situ soils.

The results suggest that during the installation of DSM columns, a positive ground effect occurs in a similar way to other methods of ground improvement. However, further research, including additional testing following this empirical method, laboratory testing and finite 2D and 3D modelling, would be useful to quantify, in detail, how in situ soils respond and how practitioners should consider these test results in their designs.

This thesis begins to evaluate how alluvial soils tend to respond to DSM. Conducting more testing on the research site, on other sites in Christchurch, and around the world, would provide a more complete data set to confirm the results of this research and enable further evaluation. Completing this additional research could help geotechnical DSM practitioners to use standardised empirical test methods to measure and confirm ground improvement rather than using existing test methods in future DSM projects. Further, demonstrating the effectiveness of empirical test methods in a DSM context is likely to enable more cost effective and efficient testing of DSM columns in future geotechnical projects.

Acknowledgements

During my postgraduate degree, I would like to thank Dr. Clark Fenton for the encouragement, direction and guidance throughout my studies. I feel so fortunate to have such a knowledgeable and supportive supervisor.

Thanks to Alan Thorp from Ground Investigation Limited for helping with the planning, investigation and test set up. Also, thanks Elizabeth Lochhead for the help and encouragement with writing and structuring this thesis.

A special thanks to Dr. Marlene Villeneuve, David Bell and Dr. Clark Fenton for the knowledge and teaching input during my undergraduate years at Canterbury University.

Thanks to all my classmates whom I shared my learning experiences with at the University of Canterbury with, you have made my university career memorable.

I am forever grateful to Graeme Quickfall, Rob Whitfield and Andy O'Sullivan for giving me the opportunity to complete this research during my time working at Hiway GeoTechnical NZ Limited. Without their encouragement and support, none of this would have been possible.

And finally thank you to my family, friends and partner. You all have supported me during my postgraduate degree and provided encouragement when I needed it most. The completion of this research is entirely attributed to your unwavering support.

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1 Introduction

1.1 Scope of Research

As New Zealand's population continues to increase, the demand for useable land for residential and commercial development has increased. Due to the topography, tectonic activity and coastal nature of New Zealand, land development and reclamation tend to extend into areas where soils are loosely compacted and prone to liquefaction (O'Sullivan & Quickfall, 2011). Liquefaction prone soils are susceptible to settlement due to post-liquefaction consolidation, lateral spreading and volume loss due to the expulsion of material to the ground surface.

In the Canterbury region (Figure 1-1), following the 2010 – 2011 Canterbury Earthquake Sequence (CES), the peak ground acceleration (PGA) from four prominent earthquake events (September 2010, February 2011, June 2011 and December 2011) caused saturated, fine-grained, granular soils in the near surface, to undergo cyclic strain softening inducing liquefaction. Liquefaction induced settlement and lateral spreading across Christchurch and its immediate surroundings caused widespread and significant damage to residential and commercial infrastructure.

The second major event within the Canterbury Earthquake Sequence was the February 2011 M6.3 event, which was the most destructive to people and property across Christchurch. In this event, 185 people were killed as a result of the centre of the earthquake being 10 km southwest and 5 km below Christchurch City Centre (Page, 2011). The PGA within the Christchurch City Centre exceeded 1.8g. The highest recorded PGA of 2.2g was recorded in the Heathcote Valley, 10 km south of the city centre.

Large amounts of liquefaction ejecta occurred throughout Christchurch, where over 200,000 tonnes of silt and sand was estimated to have been brought to the surface (Ministy of Civil Defence, 2011). A concentration of liquefaction induced settlement was observed in areas adjacent to rivers and waterways where sediments are of recent deposition and likely to be loosely compacted and saturated (Bastin, et al., 2015).

Understanding the response of soils in Canterbury to dynamic loading is fundamental to designing the safest, most efficient and cost effective foundations for the seismically-active region. The potential hazards associated with future seismic events have now been taken into account and consequently, ground remediation and foundation design are at the forefront of the Canterbury recovery and rebuild process.

In areas where liquefaction hazard is high enough to pose significant risk, methods of ground improvement are being implemented in order to minimize the hazard. As pressure for land development increases, the demand for methods to replace, improve, and remediate soft soils has become more significant.

To minimise soft soil issues under dynamic loading, deep soil mixing (DSM) is a viable ground improvement foundation method used to enhance *in situ* soils by increasing stiffness and alter soil geometries. This method, therefore, can reduce the potential of cyclic strain softening in soils and minimise liquefaction induced settlement and lateral spreading.

As a tool to aid engineers with the redevelopment of Canterbury, the Christchurch City Council (CCC), Land Information New Zealand (LINZ) and the Ministry of Business, Innovation and Employment (MBIE) have developed the MBIE Guidance documents (MBIE, 2012). These included specific methods of ground investigation. Technical classification, foundation, and structural design options which allow for various house construction methods and land damage severities.

Tonkin & Taylor, in conjunction with the CCC and MBIE, conducted a series of pilot and science trials to test various ground improvement methods and confirm their feasibility and practicality for residential redevelopment (EQC, 2013). The science trials included earthquake simulation and finite modelling to examine the ground improvement methods' potential to minimise liquefaction induced settlements from future seismic events (MBIE, 2012). DSM has been proven to be an effective option for redevelopment of the retirement care facility at the research site.

DSM is a globally recognised ground improvement method, originally established in the United States, and further developed in Scandinavia and Japan, to remediate soft and weak soils. The process involves the introduction of mechanically mixed, cement (or lime) jet grouted columns to increase the overall stiffness and confinement of a volume of *in situ* soil. To date, assessments focused on testing the strength of the DSM column elements specifically, rather than the *in situ* soils adjacent to the DSM columns.

This research aims to understand and quantify the effects that DSM columns have on near surface (potentially liquefiable) alluvial soils between the columns though the implementation of standardised invasive soil test methods.

By examining the changes in strength properties within soils around a DSM column, the ground improvement changes within soil types, and how these change with depth, conclusions can be drawn which suggest weather *in situ* soil response does occur as a function of DSM. These soil strength properties and ground improvement changes have investigated using shear wave velocity (V_s), soil behaviour and density response methods for this research.

1.2 Project Background

Canterbury, which is located on the eastern coastline of the South Island of New Zealand (Figure 1-1), was exposed to an earthquake sequence beginning with a M 7.1 earthquake in Darfield on 4 September 2010, approximately 40 km west of the City Centre. Subsequently, a series of damaging aftershocks followed causing repeated ground deformation. On 22 February 2011, a more devastating M 6.3 earthquake occurred approximately 10 km southeast of the City Centre. Due to proximity to the City, the PGA and the timing of the earthquake in February 2011, caused considerable damage to dwellings and commercial buildings occurred, and resulted in the death of 185 people.

Figure 1-1 - Location of Research: Christchurch, New Zealand (Google Earth, 2017)

Following this sequence of earthquake events, and because of the damage and human loss of life, there has been a greater need to understand the response of soils in Canterbury to seismic loading. The severity of damage has pushed geotechnical industries to study, design and refine existing methodologies and to create the most efficient and cost effective foundation options.

The potential hazards associated with future seismic events have been considered during the design process and thus ground improvement methods being at the forefront of the Canterbury recovery process.

DSM is one ground improvement method which is at the forefront of current design to minimise issues of soft, liquefiable soils. As the *in situ* ground is deep soil mixed, the properties of the soils change. Modification of the soils both within the DSM column and within the *in situ* soils around the columns tend to improve the shear strength, stiffness and alter permeability of the soil unit as a whole. Therefore, as a consequence, minimising the liquefaction potential, increasing the bearing strength and stiffness of the ground improved area.

The soil property modifications are measurable within the DSM column because of the current industry standards for quality assurance. Daily unconfined compressive strength (UCS) samples are removed and moulded into a PQ sized cylinder and tested after 7, 14 and 28 days to monitor the curing of cementitious binder and soil mixture during the construction of the foundation. Typically, trial columns are completed prior to the full construction phase, with specific testing of the columns to ensure the mix design generates adequate strength gains to meet the specification of the building scope.

Minimal research or testing has been conducted to confirm the response and modification of the soil characteristics around the DSM columns, therefore provides the justification for this research.

The objectives of this research are to examine the changes in soil characteristics by measuring the confinement radially around DSM columns and to assess whether there is a reduction in the cyclic strain softening and liquefaction potential of the saturated soils following DSM. The benefit of this is that if the lateral confinement, stiffness and shear strength of the *in situ* soil was easily quantified, and consistently demonstrated, less invasive and more cost effective testing could be used without the requirement for trial columns, providing significant cost savings, shortening the construction time and providing for efficient mix designs for future geotechnical projects.

1.3 Research Objectives

Following the CES, many ground improvement studies have been undertaken throughout Canterbury to examine methods previously not well known to geotechnical professionals. DSM was one of the processes tested for its effectiveness within near surface alluvial soils through simulation of a ULS earthquake event by detonating explosives in liquefiable soils. The research site in Shirley, Christchurch, had been exposed to high PGA during the CES and caused cyclic strain softening, liquefaction induced settlement and lateral spreading within the saturated, near surface alluvial materials.

DSM was the ground improvement method recommended as the most efficient, practical and cost effective and option to minimise the potential of future cyclic strain softening, liquefaction and lateral spreading potential during a future seismic event.

Typically, industry standard DSM quality assurance testing focuses on the DSM column stiffness and the consistency of the soilcrete following column installation rather than the *in situ* soil response to ground improvement around the columns.

Currently, the soil changes within the ground around DSM columns following ground improvement are poorly understood. The objective of this research to quantify the changes to *in situ* soil characteristics within the un-improved soils around the DSM columns following column installation.

The primary objective of this research is:

1. Assess the ground improvement of ("unimproved") soils around DSM columns within alluvial soils typical to Christchurch.

The sub objectives of the first objective are to:

a. Conduct standardised invasive field testing methods to provide background information on the selected research site and enable empirical correlations between soil parameters and behavior.

b. Conduct standardised invasive field testing methods to compare the changes in soil characteristics of “unimproved soils” prior to and following the installation of deep soil mixed columns.

c. Examine of the soil characteristic response to DSM within varying soil types common to Christchurch and how these soils vary in response with depth of column installation.

d. Determine whether there is a grouping affect or interaction between 0.9 m diameter DSM columns installed at 2.9 m centers within Christchurch alluvial soils.

e. Determine whether using standardised invasive test methods provides an accurate and repeatable approach to assure the effectiveness of DSM column installation for ground improvement.

1.4 Geological Setting

The Canterbury Plains are located on the east coast of the South Island of New Zealand. Christchurch City is bounded to the south by the abrupt topography of the Port Hills (produced by the extinct Banks Peninsula and Lyttleton Volcanic Complex), to the west by the Southern Alps (and the Alpine Fault), to the north by the Waimakariri River, and to the east by the Pacific Ocean.

The South Island is located on the transpressional plate boundary between the Australian and Pacific plates. The majority of the plate motion is accommodated by the predominantly right-lateral strike-slip Alpine and Marlborough fault systems. The contractional component of plate motion is accommodated by a series of fold and thrust zones extending from Canterbury northwards through Marlborough. The underlying basement rock is prevalent throughout the Southern Alps, mainly consists of well-indurated intensely folded sandstone and mudstones (greywakes and argillites). This material underlies the City at a depth of over 2000m below existing ground level (Brown & Weeber, 1992). The CES events were located on the latter structures. Over time, meltwater and water runoff from the Southern Alps have caused erosional processes to transport and deposit gravels, sands and silts to form large alluvial fan deposits to progressively create the formation of the Canterbury Plains (Figure 1-2).

Christchurch was chosen for settlement in 1849 as a function of the tussock covered and flat lying Canterbury Plains suggesting good potential for agriculture, horticulture and crop growth (Brown & Weeber, 1992).

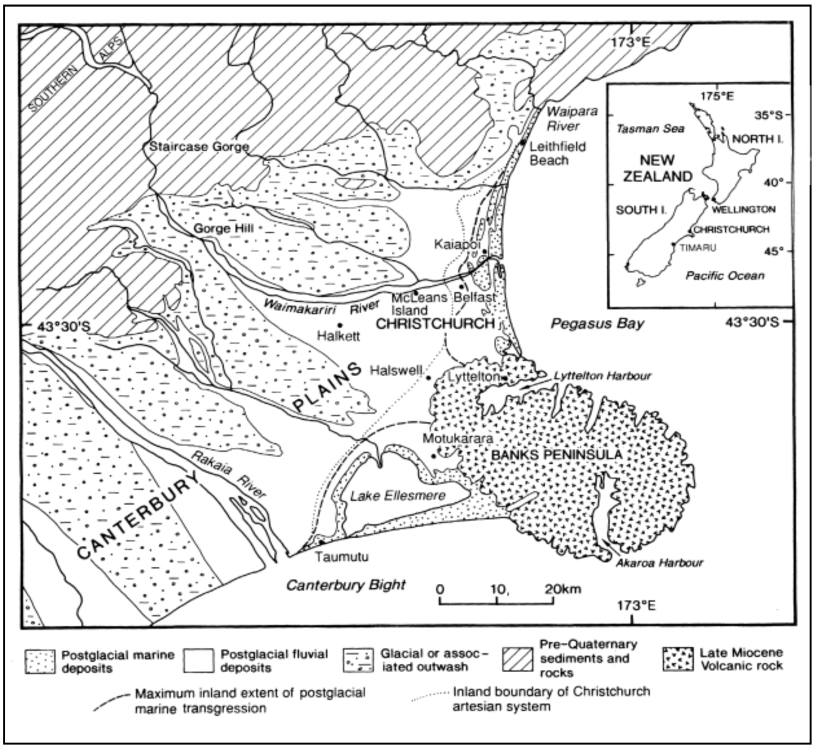


Figure 1-2 - Canterbury Plains Geology (Brown & Weeber, 1992)

A majority of the urban area of Christchurch City is located on the Waimakariri flood plain south of the Waimakariri River. The sedimentary rocks evident on the Canterbury plains are a function of the large marine transgression and regression events through the Late Cretaceous period (approximately 65 Ma to 100 Ma). The Quaternary, unconsolidated deposits evident in the upper layers underlying the coastal Canterbury Plains extend to a depth of approximately 400m (Brown & Weeber, 1992).

More recently and more relevant to this research, near the Canterbury coastline, the Quaternary, landscape evolution and geomorphology has been influenced by interglacial and glacial fluctuations, and eastern flowing river catchments that emanate from the Southern Alps (Figure 1-3).

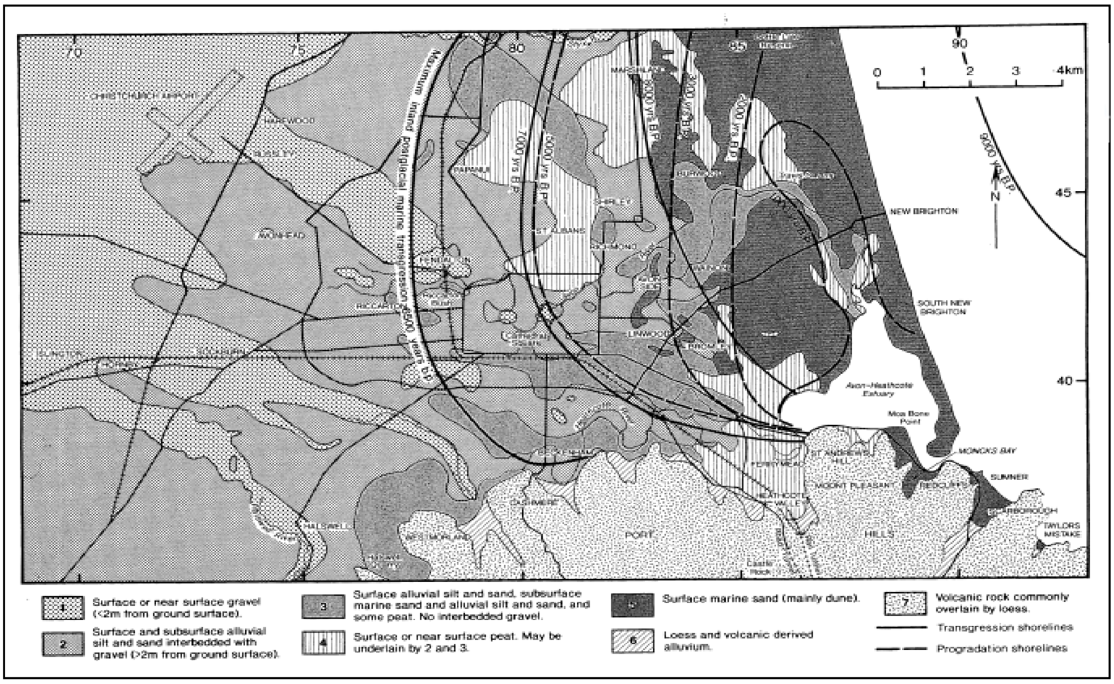


Figure 1-3 - Postglacial marine transgression across the Northern Canterbury Plains (Brown & Weeber, 1992)

Low sea levels were evident during the sea ice rich glaciation periods and high sea levels were evident during the warm interglacial periods. This caused fluctuations in sea level and subsequent sedimentation of the landscape. An inter-fingering of river gravels, with coastal sands, silts and peats became prevalent (Figure 1-4) across the Canterbury Coast line as the glacial ice advanced and retreated.

Inland valleys in the Southern Alps were eroded and shaped by large ice masses and sediments were transported to coastal and offshore reaches of New Zealand. Specifically, the current Christchurch geomorphology portrays the result of localised volcanism, sedimentation, tectonics, rock strength variation and erosion.

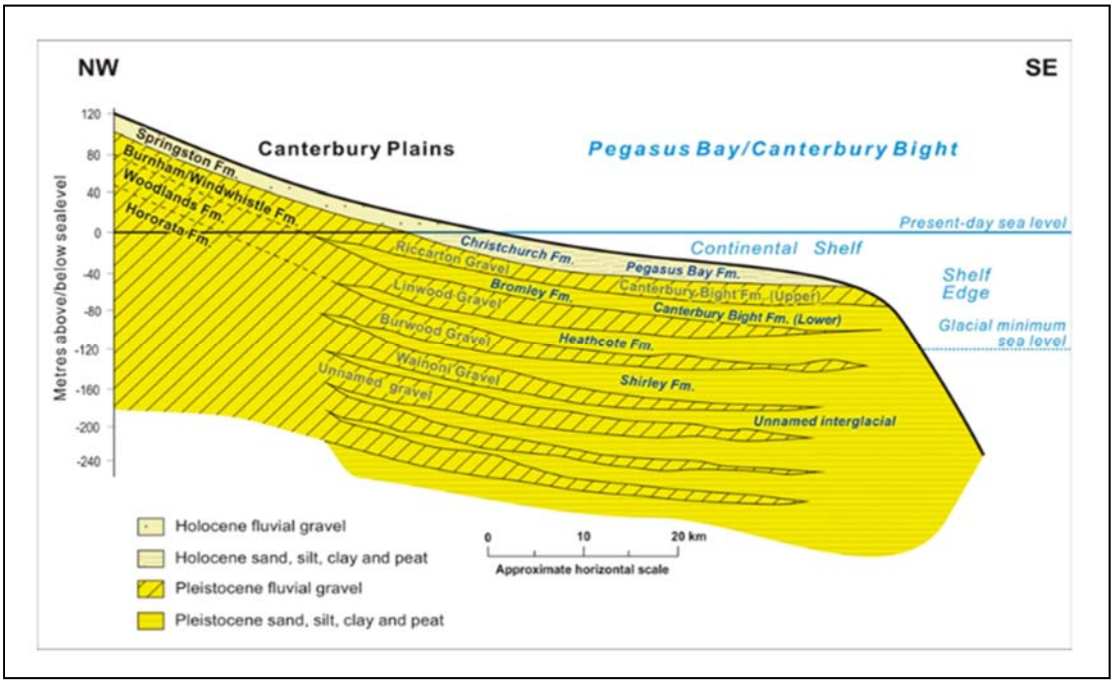


Figure 1-4 - Canterbury Planes Geological Setting with Interfingered Geological Units (Brown & Weeber, 1992)

Willowlea Retirement Village, which is located in Shirley, Christchurch (orange box indicated approximate site boundary), has been selected as the primary research site and data source for this thesis (Figure 1-5). The site is situated on the northwest quarter of Christchurch City directly adjacent to Hills Road Drain waterway indicated by the blue arrows in Figure 1-5.

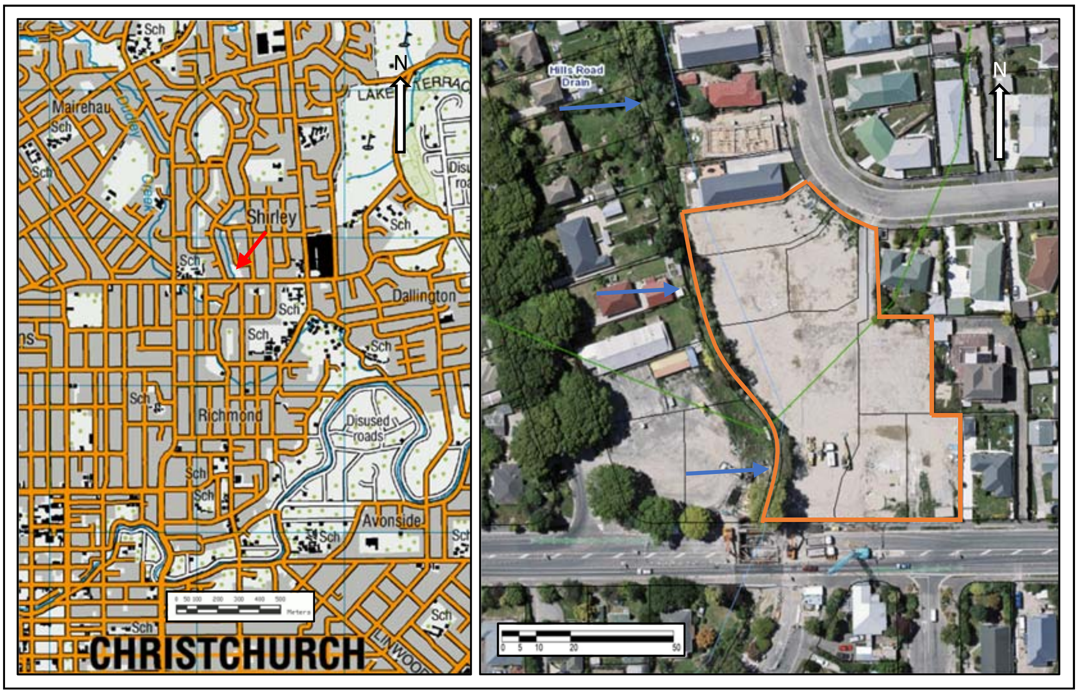


Figure 1-5 – Christchurch Waterways: Hills Road Drain adjacent to Research Site (LINZ, 2017) (Canterbury Maps, 2017)

The site is undergoing redevelopment following damage sustained by the Canterbury Earthquake Sequence. Prior to this research, site specific geotechnical investigation had been undertaken to quantify the damage onsite and provide recommendations for ground improvement. These investigations were available for review as part of this this research.

1.5 The Canterbury Earthquake Sequence

Between September 2010 and December 2011, the greater Canterbury region experienced four significant earthquakes with varying PGAs and magnitudes (Table 1).

Table 1 - Canterbury Earthquake Sequence Summary (GNS Science & EQC, 2012).

Canterbury Earthquake Sequence Conditional PGA's				
Date	04-09-2010	22-02-2011	13-06-2011	23-12-2011
Magnitude	M7.1	M6.3	M6.4	M5.9
Maximum PGA	1.26g	2.2g	2.13g	1.0g
Depth	10km	5km	6km	8km
Fatalities	0	185	1	0

The first event of the CES was the M7.1 4 September 2010 earthquake (the 'First Event') which had a maximum recorded PGA of 1.26g. This event was centred in Darfield approximately 40 km west of Christchurch City Centre and struck at 4:35 am local time. This event nucleated at 10 km depth on the previously unrecognized Greendale Fault. The earthquake event produced approximately 40 seconds of shaking.

The First Event caused surface rupture across low relief agricultural landscapes with a maximum measurable fault displacement of 5.3 m, and an average displacement of 2.3 m (Quigley, et al., 2010). The fault system was predominantly dextral strike-slip with localised discrete shears and surface bulging where the width of deformation varied from approximately 3 m to over 300 m (Van Dissen, et al., 2011).

This event caused widespread power outages and severe damage to local residential properties. Two people were severely injured by a falling chimney and broken glass.

The M6.2 22 February 2011 earthquake (the 'Second Event') had a maximum-recorded PGA of 2.2g. The Second Event struck at 12:51 pm, 10 km south west of the City Centre at a depth of 5 km. The event was felt as far north as Tauranga and as far south as Invercargill.

The Second Event was the result of slip on an 8 km fault striking east-northeast (dipping 65 degrees south) at a depth of 1 to 2 km beneath the Avon-Heathcote Estuary. The highest recorded PGA of 2.2g at Heathcote Primary School (Geonet & GNS Science, 2011) was significantly greater than expected for a M6.3 earthquake. Within the Christchurch City Centre, PGA was recorded at approximately 1.8g. These high ground motions were some of the highest recorded values in the world at the time of the earthquake.

Of the 185 people that lost their lives, 178 people died due to structural collapse of commercial and residential buildings, three people were killed by rockfall and four additional deaths were medically associated with the earthquake (New Zealand Police, 2012). It is not known how many deaths were solely attributable to liquefaction induced settlements.

Over 80% of Christchurch's water and sewerage systems were severely damaged in the Second Event. Nearly 100,000 homes were damaged requiring repair, and over 10,000 homes required demolition and to be rebuilt due to the severity of the seismic shaking and liquefaction induced settlement (Clifton, 2011).

The M6.4 13 June 2011 earthquake (the 'Third Event') had a maximum recorded PGA of 2.13g localised approximately 6 km below ground level and about 10 km east-southeast from the Christchurch City Centre (Geonet & GNS Science, 2011). However, the peak PGA in the City Centre was 0.78g which was significantly less than the PGA of 1.8g felt (in the City Centre) as a result of the Second Event. The Third Event caused one death and 50 injuries due to structural building collapse (The New Zealand Herald, 2011).

The M5.9 23 December 2011 earthquake (the 'Fourth Event') occurred at 1:58 pm and was centred 6 km east of Christchurch City off the coast of New Brighton. The Fourth Event had a maximum recorded PGA of over 1.0g (Geonet & GNS Science, 2011).

This event, along with a series of strong aftershocks, interrupted power and water supplies and caused three unoccupied buildings to collapse. However, in contrast to previous earthquakes, no injuries or deaths were recorded as a result of the Fourth Event (The Guardian, 2011).

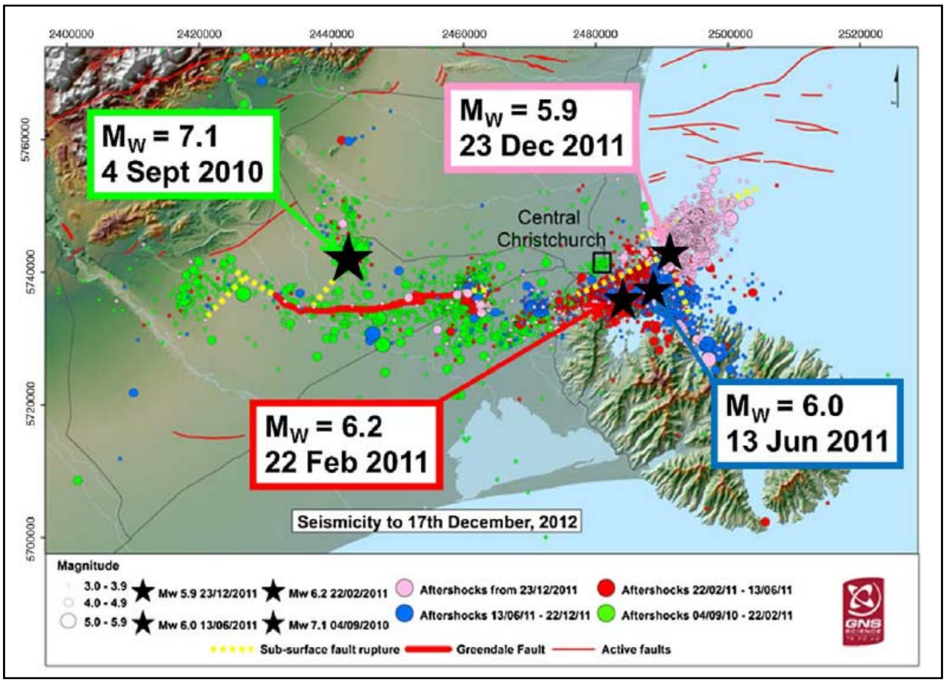


Figure 1-6 - Canterbury Earthquake Sequence Primary Earthquake Distribution (Kaiser, et al., 2012)

1.6 Christchurch Liquefaction Response

Christchurch typically displayed the most severe liquefaction induced settlement and lateral spreading in areas with loosely compacted, saturated fine soils around rivers and waterways. This is commonly due high water table and a free face to allow subsidence (Casagrande, et al., 1963).

The process of liquefaction occurs where soils are loose, moderately saturated with poor drainage and are typically granular (silty sands) (Jefferies & Bean, 2015). During seismic loading (cyclic undrained), liquefaction is caused by loose granular soils tending to decrease in volume as grain alignment collapses, causing pore water pressures to exceed the confining pressures, which results in a decrease in shear strength (and thus a reduction in effective stress) (Terzaghi, et al., 1996, Seed & Idriss, 1971, Idriss & Boulanger, 2008). Therefore, a loss in soil strength causes areas with additional normal stress (for example, building loads) to differentially settle. This process causes ejecta to be transported toward the surface and to be deposited in soil boil formations.

Areas highly susceptible to liquefaction within the Canterbury Region are those areas close to waterways and incorporated former river channels, estuaries and reclaimed land underlain by shallow (upper 10 m) alluvial soils (loose silts and sands) which are saturated (Youd & Hoose, 1977, Seed & Idriss, 1971, Orense, et al., 2012).

After the events of the CES, areas with greater risk of liquefaction induced settlement and lateral spreading were deemed too inefficient for reinstatement, or 'red zoned' by CERA (2013). In general, Christchurch properties close to a significant waterway were red zoned (Figure 1-7) due to sustaining such significant damage or were deemed Technical Classification 3 (TC3) with a clause for considerable ground improvement requirements for rehabilitation (CERA, 2013). TC3 properties are defined as likely to undergo moderate to significant land damage from liquefaction in future earthquakes of a large magnitude. Site-specific geotechnical investigation and specific engineering foundation design is required for redevelopment.

Red zones are areas of public exclusion and are deemed too greater risk to residents to allow access and habitation. GNS, EQC and the Christchurch City Council (CCC) quantified the maximum level of risk to life safety as if you had a 1 in 10,000 chance of dying, the area was red zoned (GNS Science & EQC, 2012).

Within the eastern suburbs of Christchurch City, the residential red zone refers to flat land that is subject to severe liquefaction or the related effect of lateral spreading, and is deemed uneconomical to repair. The Port Hills red zones are categorised areas which are uneconomical to protect residential and commercial property from rock fall or land that is close to a cliff edge and likely to collapse under future dynamic loading (CERA, 2013). The Central Business District (CBD) red zones are areas within the city where buildings had failed (or were severely weakened) and could cause potential harm to the public in future seismic events.

Alternatively, in areas of lower risk, green zones were defined where land was able to be remediated. These green zone sites were usually specifically investigated using a standardised investigation procedure to ascertain a technical classification (TC1, TC2 or TC3) for the ground conditions and likelihood of liquefaction damage in a future earthquake. Once classified, the severity of likely damage and potential future damage was quantified and an adequate design was produced with direction from the MBIE guidance documents.

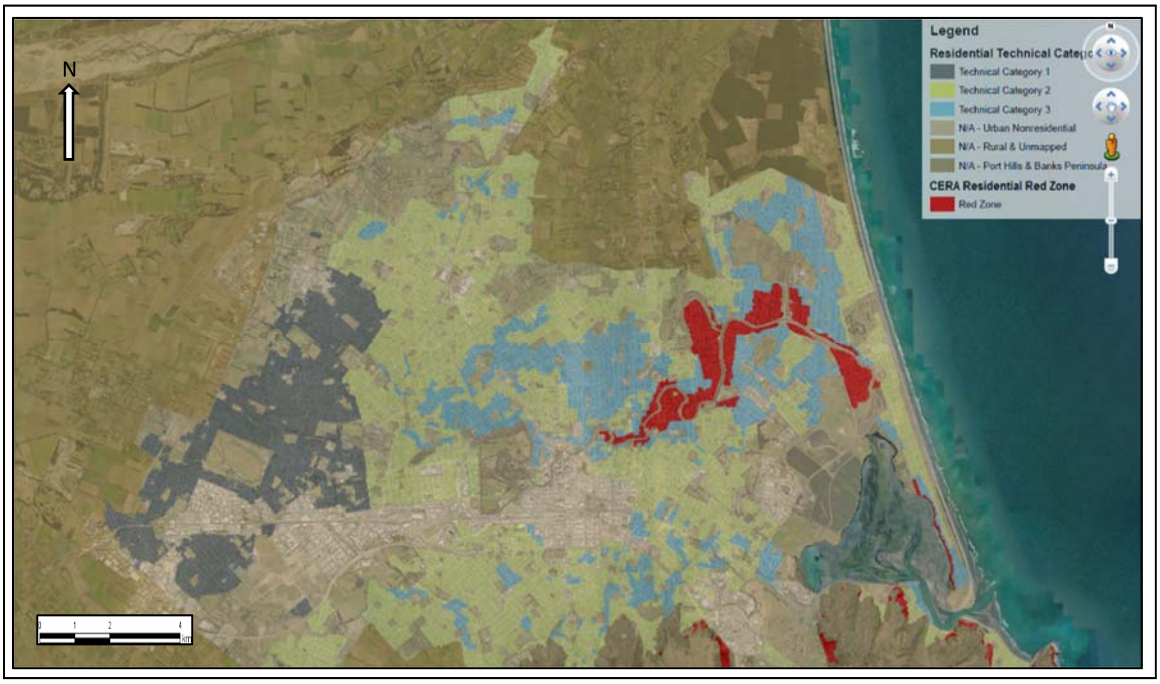


Figure 1-7 - Technical Classification across Christchurch (MBIE, 2012)

The individual technical categories for green zone development in Christchurch are summarised below (MBIE, 2012).

- Technical Category 1 (TC1 - Grey) – Future land damage from liquefaction is unlikely. You can use standard foundations for concrete slabs or timber floors.
- Technical Category 2 (TC2 - Yellow) – Minor to moderate land damage from liquefaction is possible in future significant earthquakes. You can use timber piled foundations for houses with lightweight cladding and roofing and suspended timber floors or enhanced concrete foundations.
- Technical Category 3 (TC3 - Blue) – Moderate to significant land damage from liquefaction is possible in future large earthquakes. Site specific geotechnical investigation and specific engineering foundation design is required.

1.7 Structure of Thesis

This thesis consists of seven chapters. Following this introduction (Chapter 1), Chapter 2 summarises the relevant literature review of DSM, typical soils of the Canterbury region, and existing testing standards commonly used for testing the effectiveness of the soilcrete during DSM installation, and the field testing methods selected to measure *in situ* soil response in this thesis. This research investigates the processes of liquefaction and the effects on the Canterbury region.

Chapter 3 discusses the site location, and the regional, local, and site geology associated with the research site in Christchurch. In addition, Chapter 3 summarises what happened at the test site following the CES and what the proposed remedial options for the site are.

Chapter 4 introduces the proposed testing methods used to quantify the effectiveness of DSM within Christchurch soils. Chapter 4 incorporates a detailed description of the tests that were conducted (including the apparatus used), where the tests were conducted, and how they were completed.

Chapter 5 reviews the results of the fielding testing completed at the test site in Christchurch.

Chapter 6 summarises the results of the field testing performed at the test site and compares the outcomes from the fielding testing, how soil properties changed after DSM, and responses in various soils with depth.

Chapter 7 summarises and draws conclusions from the research through discussion of the testing processes, and considers what future testing or research could be completed to form a more comprehensive data set.

2 Ground Improvement

2.1 Introduction

In current practice, DSM is used to strengthen weak and permeable soils by changing the soil's physical and mechanical characteristics through mechanical mixing and jet grouting. The process increases the soil's stiffness and shear strength which consequently minimises liquefaction potential and reduces the soil's compressibility through an open grid column arrangement.

The existing methods for quantifying DSM ground improvements involve quality assurance testing of the *in situ* mixed soils with laboratory tests focusing specifically on the soilcrete and DSM column element. However, currently there is little research summarising the (unmixed) *in situ* soil response to DSM in a Christchurch alluvial soil context.

Chapter 2 presents a review of relevant literature that has been completed summarising an overview of DSM including research on the process, applications, DSM advancements since early development of the technique, and how the altered soil characteristics are measured through existing quality assurance testing. Issues will be addressed with the current testing approach and justifications will be made on what can be explored further to answer the research objectives of this thesis.

In addition, a summary of the chosen *in situ* field testing techniques and interpretation processes used for this research are reviewed. A summary of how these methods aim to examine and answer the objectives of this research by quantifying the *in situ* soil characteristic changes in response to DSM, are also discussed.

2.2 Deep Soil Mixing (DSM) Overview

DSM is a ground improvement foundation method used to enhance the *in situ* soils by increasing stiffness and alter soil geometries. This, as a consequence can reduce the potential of cyclic strain softening in soils and minimise liquefaction induced settlement and lateral spreading.

Originally developed for environmental remediation of soils, more recently DSM applications are used to create hydraulic cut-off walls, form excavation support walls, provide improvement of weak soils, create tunnelling support, and foundation support for buildings being developed on soft soils (Hussin, 2006).

The columns are installed by using a combination of mechanical mixing and high pressure injection (jet grouting) of a cement lime slurry binder. Binders can be introduced to the *in situ* soil either in a wet (slurry) form or dry form. This research is focused on the wet mixing method installed using a TurboJet Rig (Figure 2-1) common to Hiway Geotechnical (NZ) Limited.

Figure 2-1 - Deep Soil Mixing TurboJet Rig (Quickfall, 2010)

As mixing commences, the energy begins to disaggregate the solid soil particles and define the column width within the *in situ* soil (Larsson, 2003). The installed column diameter can vary from 0.6 m to 1.5 m, and up to 40 m depth, simply by altering the adjustable mixing 'wings' of the auger and lengthening the mixing shaft (Bruce & Bruce, 2003). For the Canterbury Earthquake rebuild and recovery process, the diameter for DSM columns commonly varies from approximately 0.8 m to 1 m depending on the project specification. The columns that are the focus of this investigation are 0.9 m diameter.

The auger is equipped with discontinuous mixing paddles (or flights) depending on the probable *in situ* soil that will be encountered. Typically, a 'cork-screw-like' flighted auger is more applicable within more cohesive (siltier) materials because the soil requires more mixing surface area to induce dis-aggregation. However, a paddled auger is more affective in granular materials because the soil is more readily disaggregated and requires less surface area to complete adequate homogeneous mixing. For this research, the paddled auger was implemented (Figure 2-2).

Binder is injected using pressurized air to transport the slurry through the hollow mixing shaft, down to the mixing head, and forced out of multiple nozzles (of a preselected size) under high pressure. The pressure of the injected grout is high enough to cause disaggregation of the soils and create a homogeneous soilcrete mixture, but not substantial enough to induce grout migration outside of the defined column width.

Figure 2-2 - Deep Soil Mixing Head (Hiway Geotechnical NZ Limited, 2015)

As a consequence of DSM, the *in situ* ground will likely increase in strength, lower permeability and lower compressibility of the native ground (Tatarniuk, 2014). The soil response to DSM is a function of the existing characteristics of the *in situ* soil, the mixing method, and operational parameters and mix design chosen for construction (Bruce & Bruce, 2003).

For ground improvement using DSM, the typical arrangement of the columns is subject to the proposed building size, geometry and geology of the site. However, the column array is primarily of either a square or a triangular open grid layout similar to the DSM column arrangement shown in Figure 3. This research focuses on a triangular (or hexagonal) arrangement.

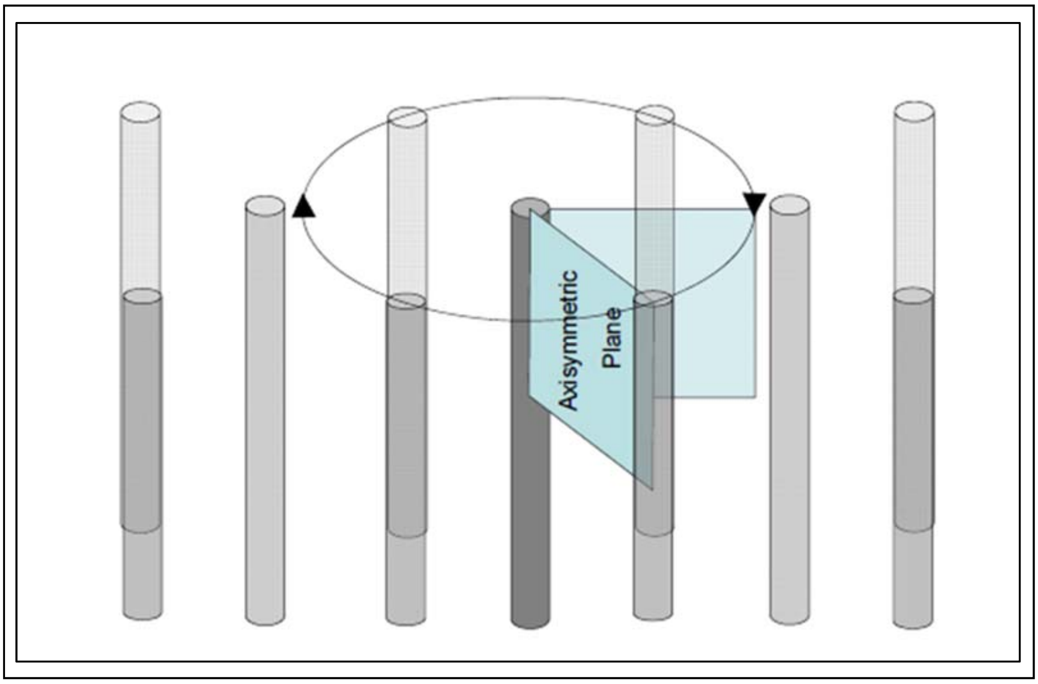


Figure 2-3 - Typical DSM Column Arrangement (Tatarniuk, 2014)

The replacement ratios of ground improvement are preselected depending on severity of the future land performance, predicted lateral spreading and liquefaction potential, and anticipated land use. These parameters are subjective to each project. However, the MBIE standards for ground improvement replacement ratios within Canterbury, are typically 16% to 20% (MBIE, 2012).

Column spacing also varies depending on project specification. However, installation of an open grid layout utilises approximately 1.8 m to 3.0 m centre to centre spacing. Terzaghi, Okada, Houghton, & Quickfall, (2005) recommend for slope stability projects, to apply a general rule of thumb of a maximum column spacing of three times the column diameter to induce a grouping effect. For flat ground improvement sites, a maximum spacing of four times the column diameter is satisfactory (Terzaghi, et al., 2005).

Column spacing is one of the main design inputs to achieve a grouping affect. If the columns are spaced too far apart, they begin to behave individually with no grouping or soil arching benefit. If the columns are too close, the overall strength gain of the improved soil system is not maximized (Tatarniuk, 2014). For this research the design column diameter was 0.9 m, with column spacing of 2.9 m (approximately three times the diameter of the column), and installation depth of approximately 8.5 m bgl.

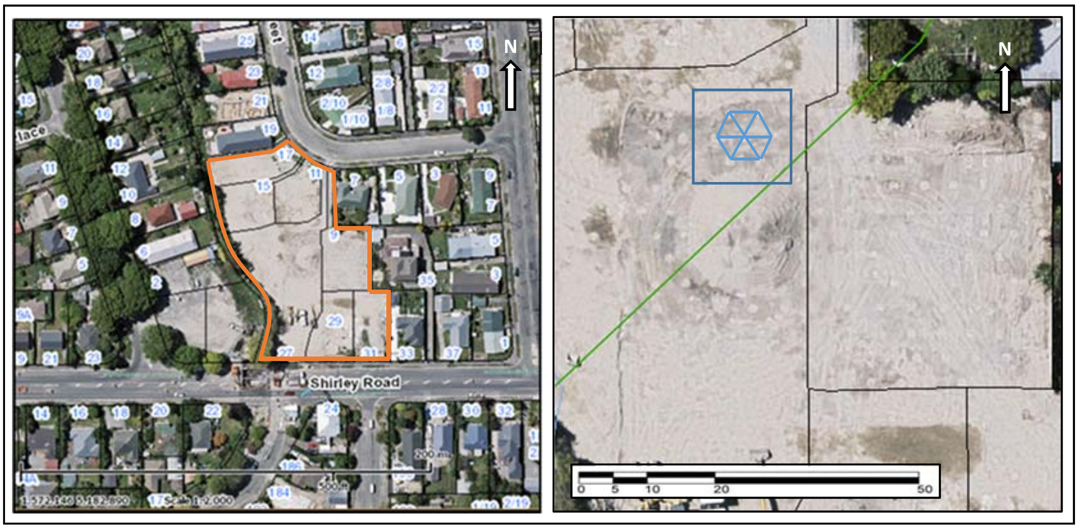


Figure 2-4 – Actual Open Grid Column Layout at the Research Site in Shirley (LINZ, 2017) (Canterbury Maps, 2017)

2.3 Outline of Existing DSM Testing Methods

DSM is a method of ground improvement via the introduction of a grout binder through mechanical mixing and jet grouting. This causes the *in situ* soils to form a homogeneous mass of soilcrete which cures (similarly to concrete) over a 28 day period. Strength gains can be in excess of 3 MPa (under unconfined compressive strength). The quality assurance testing procedures for DSM involve cement related test methods with a primary focus on column soilcrete stiffness.

The quality assurance testing is completed in three stages. During batching of the cementitious slurry binder (prior to jet grouting and mechanical mixing), the density of the cementitious mixture it tested. Immediately following the installation of a DSM column, stage two of quality assurance testing calls for a sample of soilcrete to be removed and formed into PQ sized cylinders. A sample is tested for unconfined compressive strength (UCS) at 7 days, 14 days and 28 days to monitor the curing and strength increases over time. The final quality assurance process involves coring a complete DSM column to full depth using a machine borehole at least 28 days after installation.

Current testing procedures are time consuming, invasive, and are expensive to complete. They provide specific results on how the mixed soil has improved through the introduction of a cementitious binder. However, these test method do not quantify how the *in situ* soils around the DSM columns have responded to ground improvement. Hence, why there are areas in this ground improvement method which can be further explored, and thus provides the justification for this research.

In order to answer the defined research objective, a series of cost effective, efficient and simple invasive test methods were selected to quantify the change in soil characteristic response to DSM installation. These are discussed in further detail Chapter 4 of this thesis.

2.4 Deep Soil Mixing (DSM) Mechanical Properties

Previous research has studied the effects various ground improvement methods have on *in situ* soils. Studies on the response of cohesive and cohesionless *in situ* soils to stone column ground improvement through laboratory testing has been explored by Sivakumar, et al., (2013). Muntohar and Hung (2007) have completed laboratory testing using UCS and shear vane testing of soil response to DSM columns in cohesive soils. Shen, et al., (2003) and Muntohar & Hung, (2007) have also reviewed the *in situ* soil response through the implementation of CPT testing in response to DSM.

Further studies have been conducted to examine the response clayey soils have to installed DSM columns through the use of the cone penetration test (CPT) on flat land sites (Shen & Teh, 2004), and on clay soil hill sites using the flat dilatometer (DMT) and seismic flat dilatometer (sDMT) (Tatarniuk, 2014). However, there is minimal testing and research examining DSM installation and response *in situ* soils have in a Christchurch alluvial soil context.

2.4.1 Transition Zone

Previous laboratory research has indicated a series of transition zones around the nominal diameter of DSM column in cohesive soils (Kosche, 2004). It is believed that there are three zones of influence caused by the installation of DSM columns; however, these can only be quantified accurately and economically on a laboratory research scale. These are described by Kosche, (2004), Tatarniuk, (2014) and Shen, et al, (2003) as the expansion zone (on or adjacent to the column periphery), the transition zone (or alternatively named the influential zone in some literature), and the boundary layer (Figure 2-5).

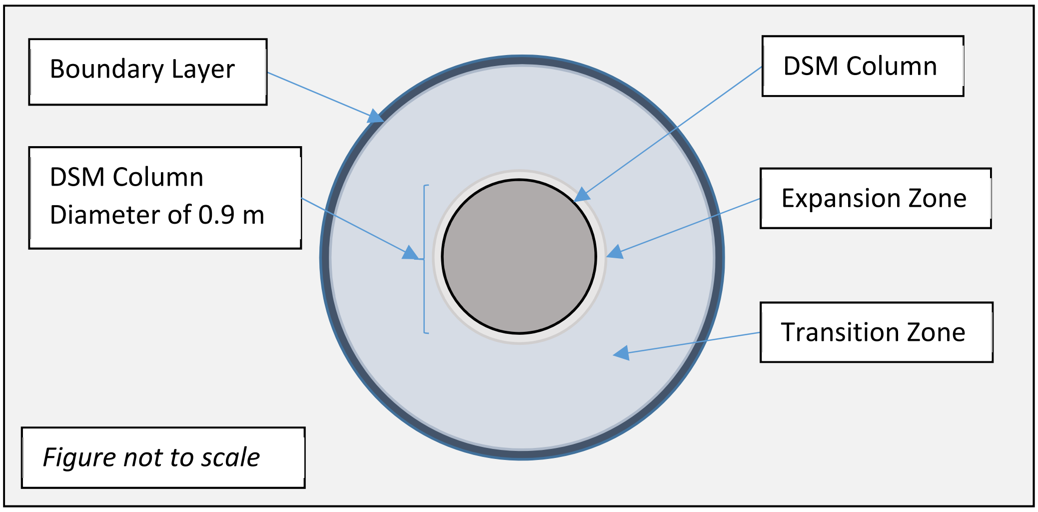


Figure 2-5 – Transition zone around DSM columns (Kosche, 2004 & Tatarniuk, 2014)

The transition zone, which is the largest of the three zones, can extend upwards of 1.5 to 2 times the DSM column diameter (Tatarniuk, 2014). For the research site, this zone could extend between 1350 mm to 1800 mm from the column centre (considering the design column diameter is 0.9 m).

The expansion zone is the difference in the diameter between the mixing blade width (subject to project design) and the actual installed column width which is defined by *in situ* soil characteristics.

The expansion zone is a function of the *in situ* soil stiffness and the pressure of the jet grouting process. In soils which are soft, coupled with a high pressure mix design, will likely cause an expansion zone to occur. However, this zone may not exist if the in situ soils are significantly stiff (Tatarniuk, 2014). The interaction of the transition zones within in situ soils around DSM columns in Christchurch alluvial soils are discussed further in Chapter 6.

2.4.2 Grouping Effects

A key design parameter for foundation and ground improvement is the performance of the open grid columns as a group. From field and project specific knowledge acquired, the efficiency of the grouping affect is dependent on several design and site-specific variables (Tatarniuk, 2014).

For ground improvement, foundation design and soil liquefaction mitigation, DSM columns are usually installed in an open grid triangular (or hexagonal) arrangement (Figure 3). O'Sullivan and Quickfall (2011) have found that the column group interaction and behaviour of the columns within varying soil materials are the most uncertain aspects of the design of DSM columns. Due to the *in situ* soils being unrestrained within the stabilised unit, soils around the DSM columns have the potential to move or mobilise when seismically loaded. DSM contracting companies often have difficulties convincing clients about the effectiveness of the designs and how the DSM columns behave as a group (O'Sullivan & Quickfall, 2011). The interaction between DSM columns and in situ soils is a key issue for DSM companies and requires further investigation in order to demonstrate the value provided by the method.

Grouping effects of piles and stone columns have been studied extensively and it has been suggested that increased efficiency of piles and stone columns as a group, has been seen when installed in an open grid array (Bransby, 1999; Chai, et al, 2007; Das 2011). Das (2011) suggests that group interaction of piles can be assumed within three to five diameters of the installed pile and Chandrasekaran, et al., (2010) assume there to be an approximate increase of 20% in maximum bending moment of soils in cohesive soils. However, in a DSM context, group interaction and response has not been studied comprehensively.

As DSM columns are installed, the process disaggregates and deforms the *in situ* soils to form a cement soilcrete column. During this process, the cement binder is injected and induces a positive volume increase in the ground. The DSM column applies lateral stresses to the *in situ* soils as consequence of DSM installation, causing confinement of the soil fabric and creating a series of zones of interaction, as described in previously (Figure 2-5).

2.5 Field Testing

Current DSM testing quantifies the effect of column installation to soils through laboratory testing of the column element as previously discussed above. However, this research aims to quantify how the soil around DSM columns responds to ground improvement in a Christchurch alluvial setting. Upon review there is little research examining the DSM installation and the response of *in situ* soils. Further, there is currently no research that focuses specifically on Christchurch alluvial soils.

This research aims to answer the research question of assessing the ground improvement of soils around DSM columns within alluvial soils typical to Christchurch by completing CPT, DMT and sDMT, which are described below in forthcoming sections of this literature review.

The proposed objectives of completing the testing is to examine if there are changes in soil stiffness, density and strength as a function of installing DSM columns in weak, liquefiable alluvial soils in Christchurch. Specific analysis on how the improvement varies with depth, changes through different alluvial soils common to Canterbury and whether a pattern is soil response around DSM columns was observed.

The *in situ* testing methods were selected for the cost effectiveness, repeatability and accuracy of the outputs generated. The CPT, DMT and sDMT, are all standardized methods that were selected to determine the soil properties of the pre and post-improvement soil profile. These methods are described in further detail below. The combination of invasive testing methods enables a collection of data to be acquired for comparison of *in situ* soil characteristic changes, stiffness and strength changes.

2.5.1 Cone Penetration Testing

Since the early development of the test in the 1930's, the CPT has become one of the most widely used tests to measure specific geotechnical properties of *in situ* soils. The CPT was initially created to examine the response of soft soils in the Netherlands, and was consequently named the Dutch Cone Test (DCP) (Robertson, 2009). However, as developments has continued, adaptions to the original method have resulted.

The layout of the CPT apparatus includes an electric measuring system embedded into a steel cone and frictional sleeve which is connected to a series of steel rods, and is advanced into the ground at a constant rate. The instrumented cone measures the resistance of the soil on the tip of the cone (q_c), and sleeve friction (f_s) at regular intervals, typically every 10 to 20 mm (Robertson & Cabal, 2014).

The system also acquires the hydrostatic pore pressure (u), the angle of the set rods and the dissipation time, which is recorded between the measurement of the overpressure obtained during the driving stage and the pressure measured when releasing the driving pressure (Pagani Geotechnical Equipment, 2017).

The cone resistance is defined by the force acting on the cone divided by the projected area of the cone.

$$q_c = {}^{Q_c}/_{A_c} \tag{2.1}$$

The corrected cone resistance for pore water effects is:

$$q_t = q_c + u_2(1 - a) \tag{2.2}$$

The normalised cone resistance is defined as:

$$Q_t = {(q_t - \sigma_{v0})}/{\sigma'_{v0}} \tag{2.3}$$

The sleeve friction resistance is defined as the frictional force acting on the friction sleeve, divided by the surface area of the sleeve:

$$f_s = {F_s}/{A_s} \tag{2.4}$$

The undrained shear strength can be estimated from the results of a CPT using the following equation:

$$C_u = \frac{q_c - \sigma'_{v0}}{N_k} \tag{2.5}$$

This test enables the determination of the soil profile and soil behaviour, interpretation of ground conditions between tests that are proximal, and the evaluation of geotechnical parameters of soils, to assess bearing capacity and correlation to other methods (Meigh, 1987).

For this research the advantages of using a CPT enables the reliable estimation of undrained shear strength, liquefaction potential, relative density, stiffness, over consolidation ratio and hydrostatic pressure of *in situ* soils. This testing method is also repeatable and provides a reliable data set (operator dependant). The testing has a strong theoretical basis for interpretation and the results can be correlated to DMT testing results (discussed in forthcoming sections).

CPT testing in Canterbury is also commonly used to examine and predict the liquefaction triggering resistance of soils (EQC, 2015). Results are inputted into computer software (CLiq) to model soil response to varying levels of earthquake shaking (PGA). This provides important information for foundation design. The CPT analysis method for analysis adopted for the Canterbury rebuild and for this research is described by Boulanger & Idriss, (2014). Although this method can tend to overestimate settlements in some soils, as suggested by Van T Veen, (2015), currently, this is the most accurate method for analysing soil response due to calibration from testing during the rebuild process.

However, this test method is limited where it cannot be used in high density soils, for example, sites with very dense sands or gravels, or site with boulders. Secondly, the sensors on the cone only provide a soil behaviour type (SBT) response, not an absolute soil measurement. So, testing must be accompanied with additional soil testing methods (like a borehole) to provide an accurate representation of the soil profile. Finally, CPT sensors can become contaminated if inadequately prepared prior to testing or the operator error in testing occurs.

Figure 2-6 - Cone Penetrometer Apparatus – 2cm², 10 cm², 15 cm² and 40 cm² (Robertson & Cabal, 2014)

Testing standards, which were followed for this research, are written by the American Society for Testing and Materials (ASTM) and are summarised in Appendix A.

The CPT test methods completed for this thesis are further explored in Chapter 4. In Chapter 5, the CPT results are used to acquire the preliminary and post ground improvement soil properties, which are compared to provide a comparison to the DMT and sDMT results.

2.5.2 Flat Dilatometer Testing (DMT)

The DMT is an *in situ* soil testing apparatus used to determine the lateral stress and soil stiffness of subsurface soils. The DMT currently is not utilised as regularly as the CPT in general geotechnical engineering practice, but can be used to provide similar results with the addition of lateral stress and stiffness. The raw results are correlated to obtain soil parameter and characteristics within *in situ* soils.

The DMT is a stainless steel blade with a flat, flexible, circular steel membrane mounted flush on one side (Figure 2-7). The blade is connected to a control unit situated on the ground surface and advanced into the ground via a pneumatic-electrical tube connection (which transmits gas pressure and electrical current) (Marchetti, 2001). The DMT blade is attached and penetrated into the ground using common invasive testing field equipment (drill rigs, CPT rigs or push rigs) (Totani, et al., 2001).

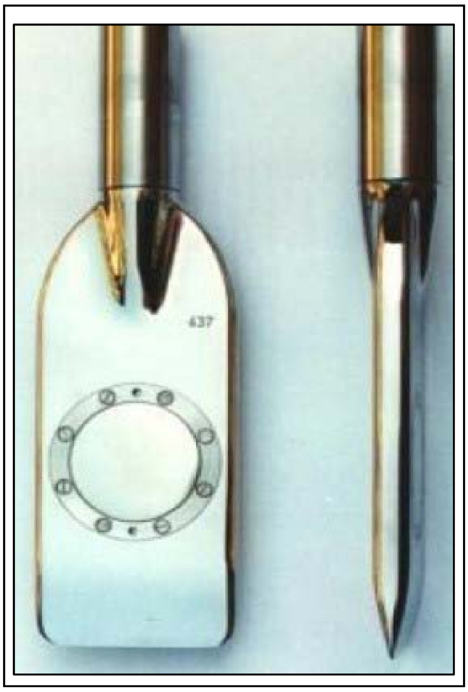


Figure 2-7 - Flat Dilatometer (DMT) (Marchetti, 2001)

The test is performed by first advancing the dilatometer into the ground, and once at desired depth, the operator then inflates the membrane and three readings are recorded. A-pressure readings measures the amount of pneumatic pressure required to move the membrane against the soil. B-pressure readings record the amount of pressure required to move the centre of the membrane 1.1 mm against the soil, and C-pressure (optional) measures the closing pressure or deflation sometime after the B-pressure reading is recorded. The blade is then advanced further (typically between 0.2 m to 1.0 m), depending on requirements and the pressure readings are completed again. The A and B pressure readings must be corrected by the values ΔA and ΔB (Δ denotes the change which considers the calibrated membrane stiffness), and are converted to p_0 and p_1 (Equation 2.6 and 2.7).

$$p_0 = A + \Delta A \tag{2.6}$$

$$p_1 = B + \Delta B \tag{2.7}$$

These principles and conversions are described in further detail in the research conducted by Marchetti (1980) and Marchetti (2001).

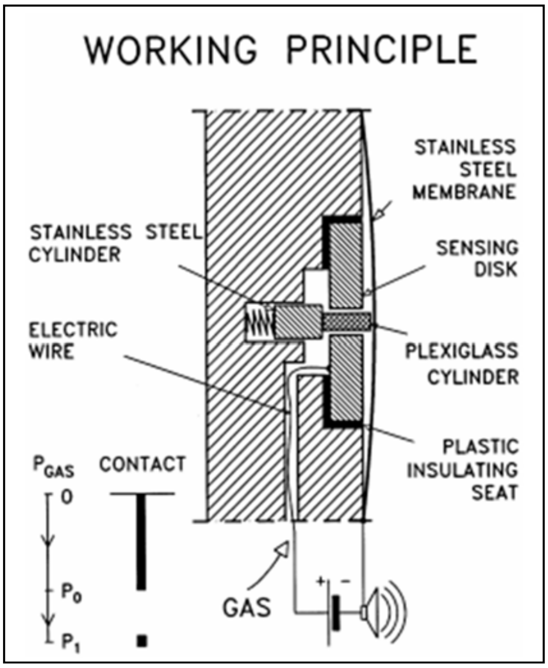


Figure 2-8 – DMT Working Principle (Marchetti, 2001)

The A and B pressure readings, once converted (to p_0 and p_1), with the pre-insertion hydrostatic pore pressure (u_0), enable the estimation of the material index (I_D) or soil type observed *in situ* (Equation 2.8).

$$I_D = \frac{p_1 - p_0}{p_1 - u_0} \tag{2.8}$$

The horizontal stress index (K_D) incorporates p_0 (and p_1), with the pre-insertion hydrostatic pore pressure (u_0) and the pre-insertion overburden stress or *in situ* vertical stress (σ'_{v0}). K_D is the key output from the DMT and is used to calculate several key parameters for soil analysis. The K_D profile is representative or a similar shape to the over consolidation ratio (OCR) (Marchetti, 1980).

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \tag{2.9}$$

The undrained shear strength, which is defined by the *in situ* vertical effective stress (σ'_{v0}) and the horizontal stress index (K_D) is estimated below.

$$C_u = 0.22\sigma'_{v0}\,(0.5K_D)^{1.25} \tag{2.10}$$

The dilatometer modulus (E_D) is derived from the theory of elasticity (Gravesen, 1960) and relating to p_0 and p_1, for a 60 mm diameter DMT membrane creating 1.1 mm of displacement is found by:

$$E_D = 34.7\,(p_1 - p_0) \tag{2.11}$$

The five key parameters of the DMT suggested above, can be used to interpret the over-consolidation ratio (OCR), coefficient of lateral earth pressure at rest (K_0), undrained shear strength (c_u) and the vertical drained constrained modulus (M). For further detail on the basic DMT reduction formulae, see Appendix B for full information.

Marchetti (1980) recommends that the pre-insertion pore pressure (u_0) and the pre-insertion vertical effective stress (σ'_{v0}) should be known to some extent prior to advancing the blade into the ground (Marchetti, 1980). Testing standards, which were followed for this research, are in accordance with ASTM and are summarised in Appendix A.

Similarly to the CPT, the DMT test is a fast, simple and robust, near continuous, repeatable and economical method for testing *in situ* soils (Robertson, 2015). However, this method relies on correlative relationships and calibration from local geologies, and is difficult to advance into dense granular materials. Fortunately, as a function of the earthquake rebuilding process, significant testing data has been recorded in Christchurch to provide sufficient testing calibration.

In Chapter 5, the DMT results are used to acquire the preliminary and post ground improvement soil properties, which are compared to the flat dilatometer and seismic flat dilatometer results.

A common adaptation to the DMT is the seismic flat dilatometer (sDMT) which incorporates the standard DMT blade with a seismic module to measure the shear wave velocity (V_s). This is discussed further in Section 2.5.3 below.

2.5.3 Seismic Flat Dilatometer Testing

Seismic testing methods measure the speed seismic waves propagate through a soil medium, from which, the small strain modulus can be calculated (Tatarniuk, 2014).

Seismic waves travel broadly in two ways; compression (P-waves) and shear waves (s-waves). Compression waves travel faster (between 1 to 14 km/s), in a longitudinal direction and at a higher frequency through all types of materials. Where shear waves travel slower (1 to 8 km/s), in a transverse direction, and will only propagate through solid materials so ground water will not affect the travel (Gelius, 2007).

The speed of travel of both P and S waves is governed by the medium the waves are passing through. Since the propagation of shear waves is slower, these can be measured with greater accuracy and the time interval between waves is greater.

The sDMT is an adaptation to the traditional "mechanical DMT" introduced and developed by Marchetti (1980) with a seismic module fitted above the DMT blade. The module is a cylindrical element, which has two receivers spaced at a 0.5 m interval to measure shear wave velocity (V_s) as s waves propagate through soils (Marchetti, et al., 2008).

Shear Wave Velocity (V_s) is derived by the difference in the distance between each of the seismic geophone receivers ($S_1 – S_2$) and the arrival delay period from the initial impact receiver (Δt).

$$V_s = (S_2 - S_1)/\Delta t \tag{2.12}$$

Typically, V_s measurements are recorded at 0.5 m intervals as the DMT advances into the ground. For this research 0.5 m testing intervals were completed.

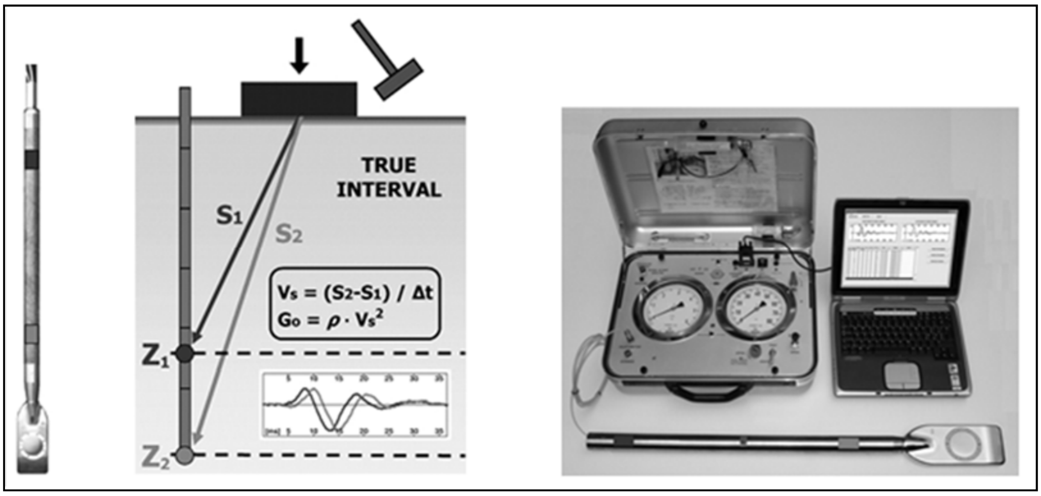


Figure 2-9 - Seismic Flat Dilatometer (sDMT) (Marchetti, 2001)

The shear wave source at the ground surface is a pendulum hammer which strikes a steel plate horizontally. The steel plate is pressed against the *in situ* soil by the weight of a rig or truck. The plate is orientated with the long axis facing parallel to the receivers, in order to generate the most effective shear wave propagation.

The purpose of shear wave testing is to measure the speed of S-wave travel through various soil mediums. Shear waves travel at very low strains, and thus, shear strain (ε_q) can be estimated from the vibration velocity of the source, and the V_s.

$$\varepsilon_q = \frac{x}{V_s} \tag{2.13}$$

The maximum shear modulus (G_0) at small strains may be determined from V_s and the bulk density of the soil medium the shear waves are propagating through (ρ) (Hird & Chan, 2005).

$$G_0 = \rho V_s^2 \tag{2.14}$$

As the waves propagate through the soil, the two geophones within the seismic module mounted above the DMT blade record the time delay and transmit the information to the computer located on the ground surface. As previously stated, there two geophones at 0.5 m spacing, which require re-phasing of the seismographs to enable the calculation of the time interval between seismic wave arrivals (Tatarniuk, 2014). Once the time interval, distance to seismic module and re-phased seismograph information is known, the shear wave velocity (V_s) can be determined.

The sDMT, being an adaptation to the DMT comes with its advantages and disadvantages. The method is a fast and economical way of testing V_s through a non-destructive process. This testing provides a theoretical basis for interpretation and is applicable for both soils and rocks. However, there is no direct sampling and the models are assumed for interpretation. Previous research suggests that the sDMT test can be affected by clays and cemented geologies, although within the Christchurch alluvial soils, these characteristics are unlikely to be observed and thus unlikely pose any risk for adverse results.

2.6 Conclusions

Existing methods for DSM quality assurance testing quantify how DSM is improving soils which are being mechanically mixed and jet grouted to form a cementitious soilcrete mixture. However, there are no common testing processes to quantify the *in situ* soil response to DSM column installation. Therefore, the objective of this research is to quantify the ground improvement response of *in situ* soils around DSM columns in a Christchurch alluvial soil context using standardised invasive testing methods.

Specific invasive field testing methods were chosen to address how the *in situ* soil characteristics are changing as a function of DSM. The CPT, DMT and sDMT tests were selected and the theoretical interpretations were summarised. The tests are fast, cost effective, repeatable and comparable, and are standardised by the ASTM.

The research will contribute to the geotechnical field by benefitting DSM practitioners by potentially enabling the use of standardised invasive methods to provide quality assurance of their product. This will provide an increase in efficiency of foundation design, quality assurance testing and enable project completion in a timelier manner.

As previously discussed, pile designs generally allow for a 20 % increase in soil benefit and soil bending moment. If a similar principle could be demonstrated with DSM, the column spacing could likely be refined from current standards. By quantifying the group effect of DSM columns would enable practitioners to increase the column spacing and result in a more efficient foundation design.

Laboratory testing methods and machine boreholes are costly and time consuming processes. Currently, existing DSM quality assurance testing incorporates both UCS and PQ borehole testing. If the response of the *in situ* soils were accurately conducted, in field testing methods could place more emphasis on soil response and less focus on soilcrete homogeneity, thus minimise cost and create time savings for both the client and the contractor.

However, the effectiveness of these revised test methods to potentially quantify the changes within *in situ* soils is entirely a function of the soil profile. The forthcoming chapter (Chapter 3) summarises the site description and site geology described at a local and site specific level.

3 Site Description

3.1 Introduction

This chapter summarises the site description including the research location, anticipated land use, and the local and site mapped geology. A brief summary of the investigation test methods used to examine the existing *in situ* soil profile is explained, and a definition provided for the *in situ* soils observed in the investigation. In addition, Chapter 3 summarises the earthquake response estimated at the test site following the CES. Finally, Chapter 3 outlines the proposed *in situ* soil testing methods used to quantify the in soil response to DSM column installation, with the aim of answering the research objectives discussed in previous chapters of this thesis.

3.2 Summary of Proposed Testing Methods

To assess and explore the areas identified where further research is required for *in situ* soil response around DSM columns, a series of field testing methods have been implemented to examine the research objectives. The following points include a summary of what has been completed:

- In situ field testing methods (hand augers, scala penetrometer tests, test pits, machine boreholes, and CPTs) were carried out to acquire soil parameters of the site. These methods and outcomes are described in subsequent sections of this chapter.
- The predicted soil response to dynamic loading was analysed to provide a baseline for the research site.
- Fielding testing (CPT, DMT and sDMT) was conducted prior to and following the installation of DSM columns, as a method to examine the potential soil property changes around DSM columns.

Chapter 4 describes these test methods in further detail, Chapter 5 summarises the results of the tests, and Chapter 6 discusses and interprets the results from the test methods.

3.3 Site Location and Land Use

The site selected for this research is located at 27 Shirley Road, Shirley, Christchurch. The site has been commercially occupied as a retirement facility on a relatively flat section of land, approximately 3800 m^2 in area (Figure 3-1).

The site is bounded by residential properties on the northern and eastern boundaries, Shirley Road to the south, and Hills Road Drain to the west. Hills Road Drain is a small, narrow waterway which feeds into the Avon River approximately 2.5 km downstream. The stream has a free face of approximately 2.0 m and a width of approximately 3 to 5 m depending on the location along the site boundary.

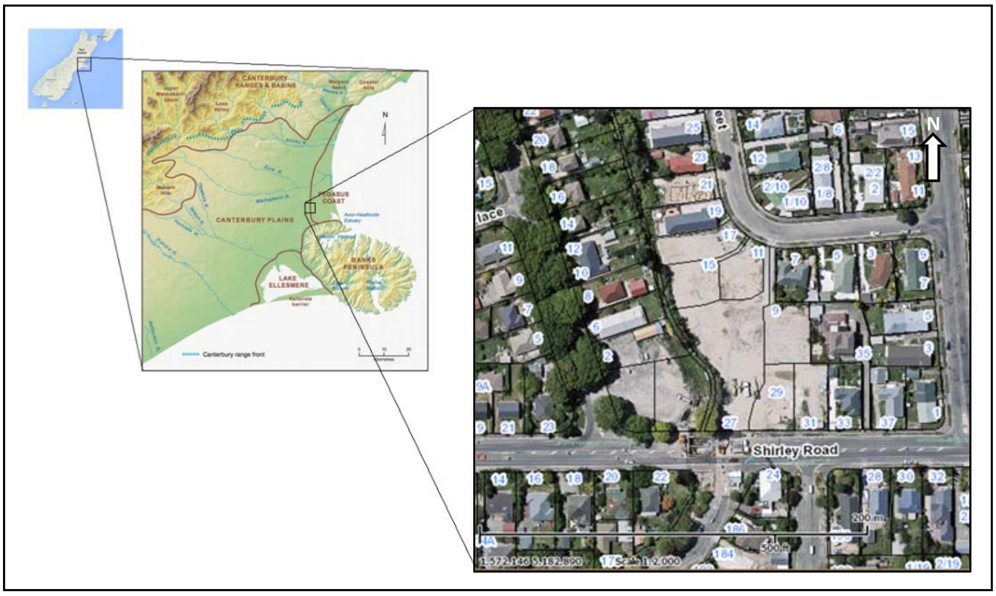


Figure 3-1 - Site Location (Google Maps, 2017)

The existing retirement care facility, which occupies a majority of the site, is likely to expand during the rebuilding of Christchurch following the CES. The proposed redevelopment involves the purchase of additional residential property to the north and southeast of the site, expanding the size of the site to approximately 5,500 m².

3.4 Desktop Study - New Zealand Geotechnical Database (NZGD)

The site had been broadly classified as TC3 (MBIE, 2012). However, prior to redevelopment, site specific geotechnical investigation was conducted to confirm the existing soil response and technical classification. The geotechnical investigation was completed prior to this research and was available for review. Figure 3-2 portrays a map of the broad technical classification of the site (with the approximate site location indicated by the red box).

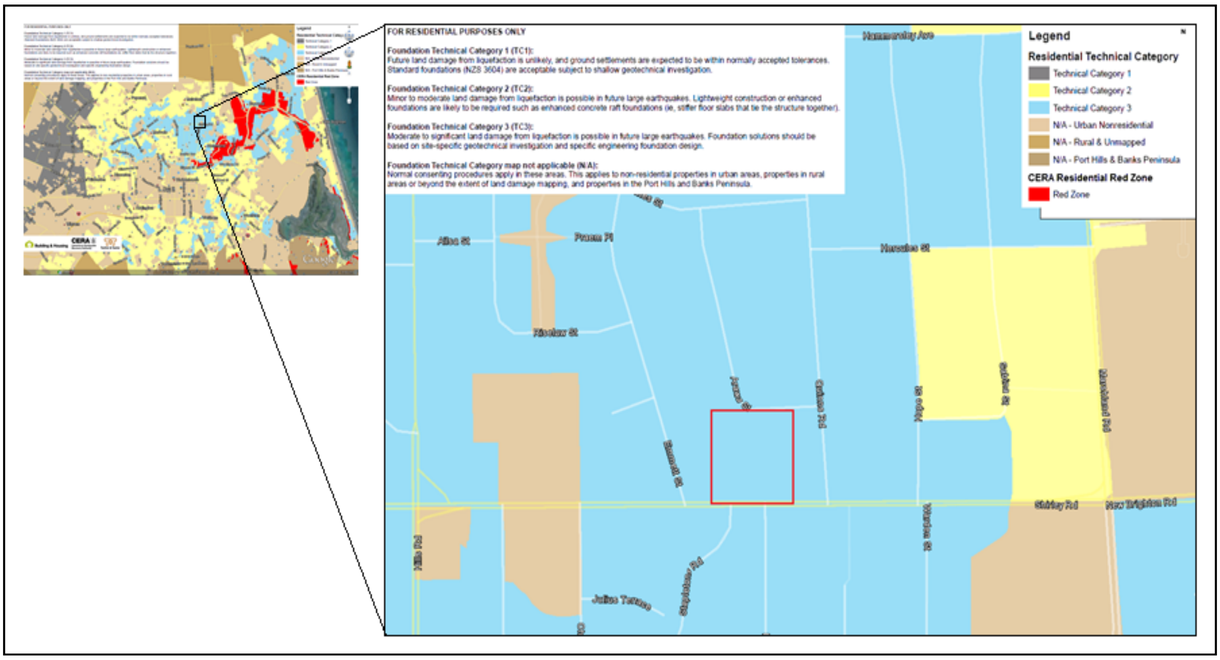


Figure 3-2 - Site Technical Classification (NZGD, 2017)

The site had been exposed to varying levels of PGA from all four events in the CES. Using Bradley, (2012) "Conditional Peak Ground Accelerations Map", the PGA from each prominent earthquake event can be estimated specifically for the research site in Table 2.

Table 1 - Summary of Earthquake Specific Data (NZGD, 2017 & Bradley 2012)

	4 Sept 2010 (Mw 7.1)	22 Feb 2011 (Mw 6.2)	13 Jun 2011 (Mw 6.0)	23 Dec 2011 (Mw 5.9)
Max PGA in Christchurch (g)	1.26g	2.2g	2.13g	1.0g
Mean PGA at site (g)	0.19	0.36	0.21	0.26
PGA Probable Range at site (g)	0.13 to 0.25	0.25 to 0.49	0.14 to 0.28	0.17 to 0.36
PGA 7.5 at site (g)	0.17	0.26	0.14	0.17

With publically available aerial photography, a visual interpretation of the site with interpretation of ground response to each earthquake event during the CES was able to be completed.

Following the September 2010 M7.1 event, using the aerial photography, the site was observed to have no liquefaction ejecta or lateral spreading. The site had been exposed to the highest PGA during the February 2011 M6.2 event (approximate PGA of 0.36 g) and, as a result, was observed by EQC to have moderate to major lateral spreading and moderate to severe liquefaction ejecta both on site and on the road. A series of 50 mm to 200 mm ground cracks were mapped in an orientation to the north-south direction aligned parallel to the adjacent waterway (Hills Road Drain) by EQC, 2011. The June 2011 M6.0 event was interpreted by EQC to have no lateral spreading but moderate to severe liquefaction ejecta on the site and on the road adjacent to the site. From interpretation of the aerial photography, moderate liquefaction ejecta was observed on site and moderate to severe liquefaction ejecta was observed on the road. As a result of the December 2011 M5.9 event and using aerial photographs, moderate liquefaction ejecta was observed on site and moderate liquefaction ejecta was observed on the road.

Using nearby data from GNS Science and EQC, the groundwater mapped at the site was recorded approximately 3 m to 4 m below ground level. The data was recorded from a nearby well approximately 150 m northwest of the site. These recorded groundwater depths are consistent with the site investigation results previously undertaken.

The historic aerial photography of the site was reviewed and relevant observations of the site summarised changes from 1941 to the present time. Preceding the earliest photography (1941), a dwelling was built, and the surrounding land in the area appeared to be agricultural. Between 1994 and 2004, the retirement village was built. There was little evidence of any placed fill or dumped material when the site was developed.

The site experienced lateral spreading and liquefaction induced settlement following the CES. Although the site is classified as a commercial building, and formerly there are no predicted settlement or lateral spreading criteria provided for construction, upon regional classification, the site has been categorised as a TC3 classification for repair or rebuilding. From review and analysis of the site investigation data, the CPT's predicted settlements in excess of 100mm laterally and 100 mm vertically during an ultimate limit state (ULS) event. The data confirms the site has performed poorly and is likely to perform in a similar manner to the predicted settlements for MBIE residential TC3 design (MBIE, 2012).

3.5 Regional Setting

To provide, understand and establish a context for the site geology, it is important to examine the broader geological evolution of the Canterbury region. This section provides a brief summary of the main stratigraphic units observed in Christchurch with commentary on the main geomorphological features present in the region.

3.5.1 Geomorphology

Christchurch City is situated on an outwash plain south of Pegasus Bay which is aggrading east toward the coastline. The city sits on overbank flood channel deposits derived from the Waimakariri River, Holocene coastal deposits and the Port Hills volcanic complex.

The eastern side of the city comprises of estuarine, lagoonal and swampy deposits as a function of the eastern aggrading coastline. The coastline continues to advance due to sedimentation and transportation of sedimentary deposits being transported downstream over the past 6,500 years.

The alluvial flood and over bank deposits from the Waimakariri River generally comprise of silts, sands and gravels, which are prevalent in the western side of the city and gently slope toward the east. The Avon, Heathcote and Styx Rivers intersect and meander through the city in a west to east direction (toward the ocean). The Avon River and Heathcote River both drain into the Avon-Heathcote estuary catchment on the eastern side of the city. The Styx River drains into the Brooklands Lagoon to the northern side of the city.

The Port Hills, which sit to the southeast of the city, are derived from the Lyttelton volcano complex. These volcanoes are part of the volcanic rim of Banks Peninsula, and were formed approximately 5.8 to 12 million years ago.

3.5.2 Geology

Brown and Weeber (1992), suggest that Christchurch City is underlain by alluvial over bank deposits that consist of silts and sands. These are some of the most recent deposits and are defined as the Yaldhurst member of the Springston Formation (Figure 3-3). Secondly, there is the Christchurch Formation which consists of overlying fixed or semi fixed sand dune deposits. These formations are underlain by the Riccarton gravels which provide the permeable pathway for artesian aquifer systems beneath Christchurch City.

The Yaldhurst Member of the Springston Formation includes river channel derived sands and gravels (fluvial), and overbank silts and sands derived from the braided river system which once flowed, and has since drained the Christchurch Area (Tonkin & Taylor Limited, 2011). The Christchurch formation consists of fixed or semi fixed sand dune deposits, which are likely a function of coastal processes. Typically, these deposits consist of fine to coarse clean sands. However, the Christchurch Formation is known to include gravels, silts and shells. The dunes are only defined as semi fixed due to their mobile depositional process that is a function of their proximity to the coastline. The Riccarton Gravels were deposited in the Quaternary Period as a function of glacial outwash processes. These are presently at depths approximately 20 to 30 m below Christchurch City.

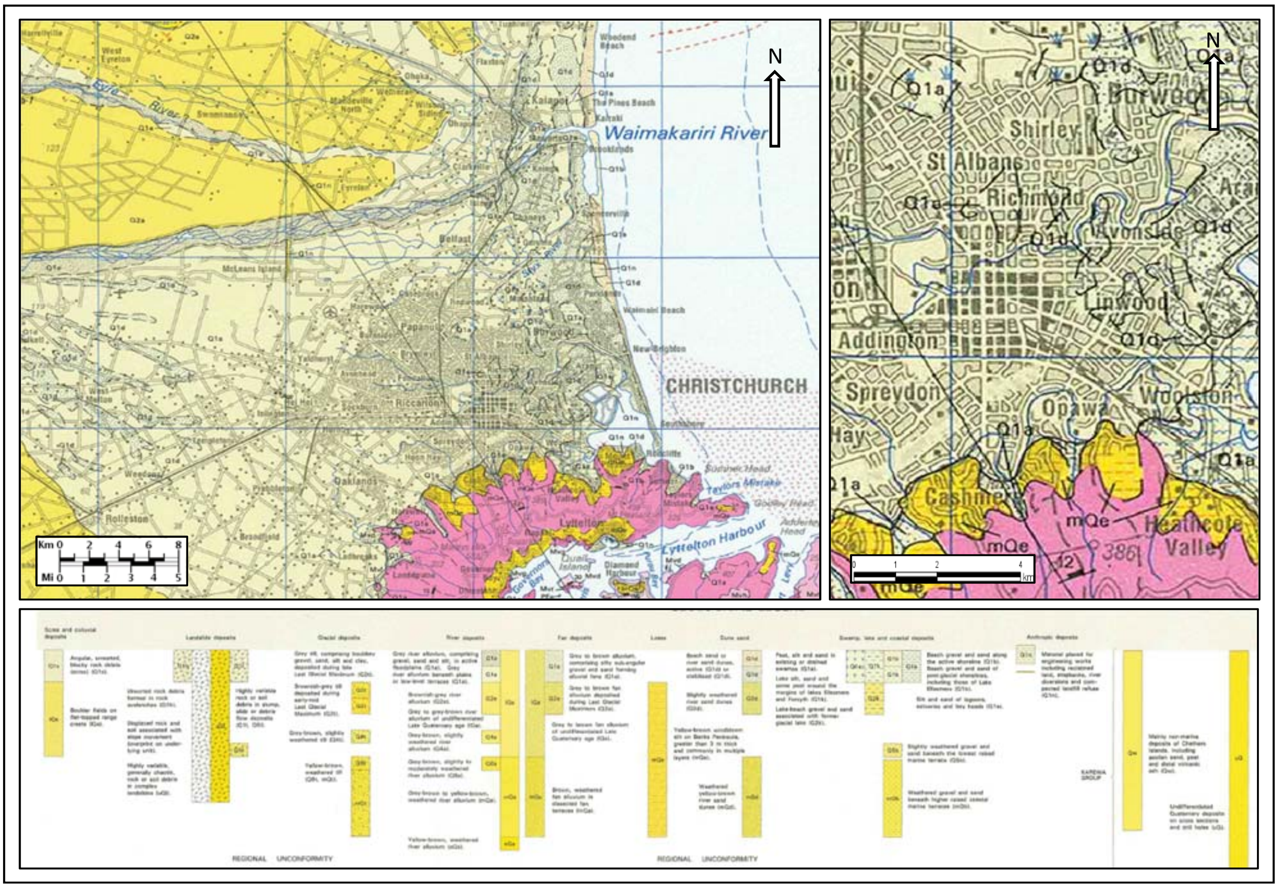


Figure 3-3 - Geological Map of Christchurch – QMap (scale 1:250,000) from GNS (Forsyth, 2008)

3.6 Recent Ground Investigations

The completion of a thorough site investigation and the collection of soil profile test data and soil samples from the site enabled the analysis of *in situ* soil conditions and the design of the most efficient foundation options. This was undertaken through a range of ground testing to various depths prior to this research, and was available for review. The results of the soil profile enabled a comparison of the soils to the broadly mapped geology and geomorphology (Brown & Weeber, 1992).

Hand augers, Scala penetrometer tests, test pits, machine boreholes (with standard penetration tests (SPT)) and Cone Penetration Tests (CPT's) were undertaken to investigate the ground conditions of the site. This section provides a brief overview of the investigations completed for the proposed re-development of the retirement facility.

3.6.1 Hand Auger and Scala Penetrometer Tests

A total of three hand auger boreholes and ten Scala penetrometer tests were completed across the site. The hand augers terminated at a target depth of approximately 3 m bgl, and the Scala penetrometer tests terminated at a target depth of 1.5 m bgl. The hand auger boreholes were completed and logged by TM Consultants Limited in accordance with NZGS, (2005) standards.

The tests were completed across the site to provide an indication of the near surface soil strength and characteristics within the upper 3 m. Standing water was not encountered during testing.

A summary of the soils observed using the hand auger bore holes is discussed below in Section 3.7 and the full test logs are provided in Appendix C.

3.6.2 Test Pits

Three test pits were completed using a 14 t excavator to a maximum depth of 4.2 m bgl. The excavations were terminated due to practical refusal because standing water was encountered approximately 3.1 m bgl. This standing water was consistent across all three test pit locations. However, the water only flowed into the test pits at approximately 3.5 m bgl below the silty, less permeable soils observed in the upper 3.5 m. Once the artesian water flow had minimised, the residual standing water depth was recorded as 3.1 m bgl.

The excavations were completed in three different areas across the proposed retirement building footprint to provide the most accurate representation of the upper site geology at the testing site. Following the completion of excavation, the test pits were logged and photographed by Coffey Limited in accordance with NZGS, (2005) standards.

A summary of the soils observed in the test pits completed with both logging of the soils are provided below in section 3.6 and the full logs are attached in Appendix D.

3.6.3 Machine Boreholes

A total of three boreholes were completed at various locations across the site. These were advanced into the ground using a roto-sonic vibratory method to obtain an HQ (96 mm outside diameter) core to a maximum depth of approximately 20 m.

The boreholes were conducted by ProDrill Limited in order to understand the stratigraphy and nature of the upper soil zones within the proposed footprint of the retirement care facility.

Standard Penetration Tests (SPT) were conducted at 1.5 m intervals during the advancement of the boreholes in accordance with the testing standards described in ASTM D1586-11 (ASTM, 2011). Following the completion of drilling, the core was logged and photographed by Coffey Limited in accordance with NZGS, (2005) standards.

A summary of the soils observed in the boreholes completed is discussed below in section 3.7 and the full logs are provided in Appendix E.

3.6.4 Cone Penetrometer Tests.

Cone Penetration Tests (CPT) with the additional measurement of pore water pressures (u) were completed at five locations across the site. The CPT tests were completed using a Pagani TG63-150 1.5t hydraulic rig, operated by McMillan Drilling Limited. These CPTs were conducted to approximately 12 m to 15 m bgl to assess the SBT and predict the liquefaction potential (further summarised in Chapter 5) of the *in situ* soils in accordance with the testing standards of ASTM D5778-12 (ASTM, 2012).

Following the completion of the testing, the SBTs were estimated by McMillan Drilling using the methods of Robertson & Campanella (1986). Further analysis using CLiq was completed to provide a comparison of the SBT estimates, using the methods discussed by Robertson & Campanella (1986) against the outputs derived from the methods discussed by Boulanger & Idris, (2014).

A summary of the soil behaviours observed in the CPTs completed are provided below in section 3.7 and the full logs are attached in Appendix F.

3.6.5 Testing locations

The site investigation test locations relative to the retirement facility footprint are depicted below in Figure 3-4.

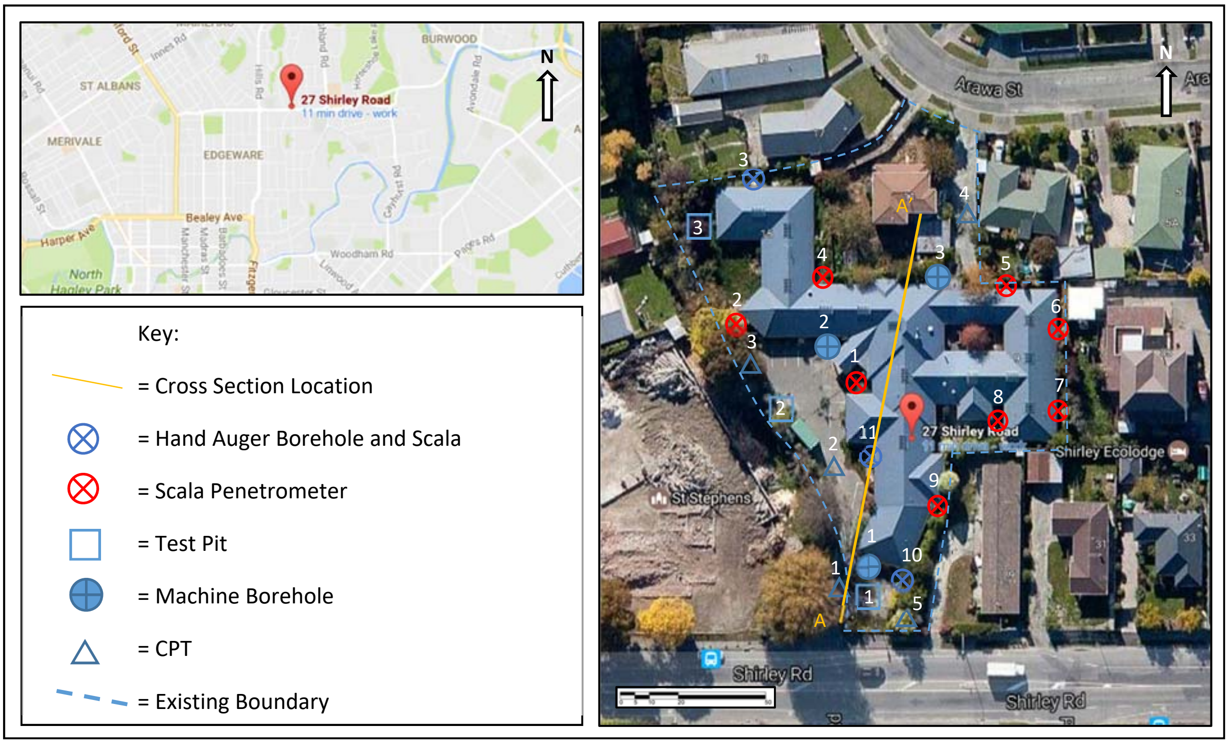


Figure 3-4 – Geotechnical Testing Locations (Google Maps, 2017)

3.7 Site Geology

The site geology is defined as "river alluvium, comprising gravel, sand and silt in active flood plains (Q1a) or grey river alluvium beneath plains or low level terraces (Q1a)" by Brown & Weeber, (1992) and Forsyth, (2008). The research site location is indicated by the red arrow and dot in Figure 3-5 below.

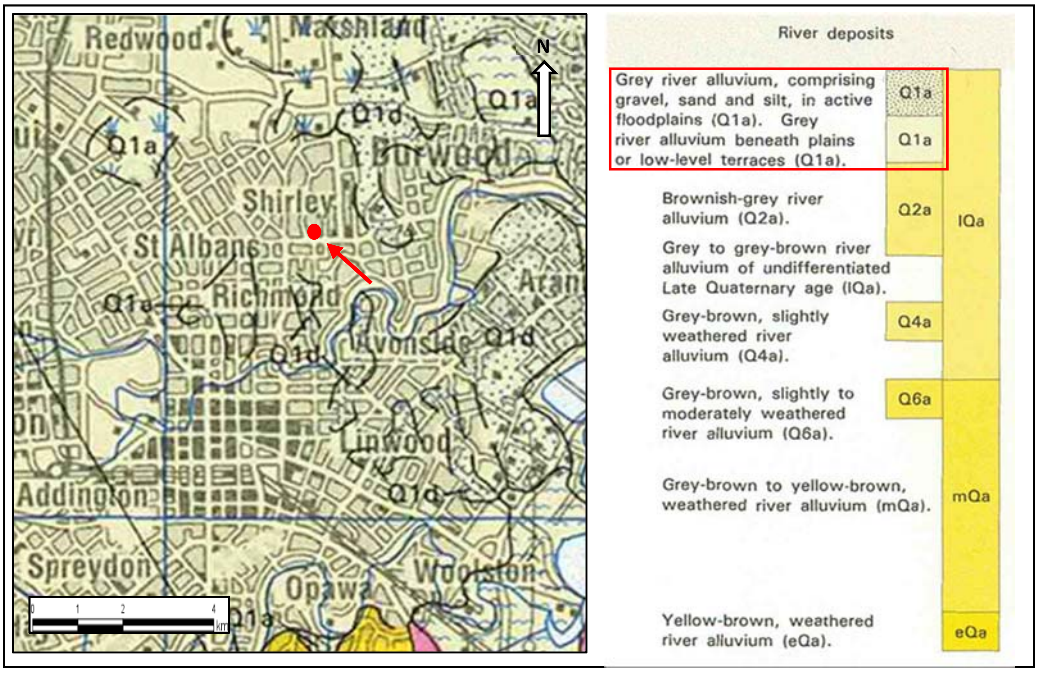


Figure 3-5 - Site Geology – QMap (scale 1:250,000) from GNS (Forsyth, 2008)

From the geotechnical investigation testing completed at the research site, the soil profile consists of interbedded silts and sands likely derived from the overbank flood deposits of the Yaldhurst member of the Springston Formation. The soils observed during the site investigation are broadly consistent with the published mapping by Brown & Weeber, (1992) and Forsyth, (2008). A summary of the site-specific soil conditions is set out in Table 3 below.

Table 2 – Summary of Site Specific Soil Conditions

Depth (m)	Soil (Behaviour) Type	Constancy / Density
0.0 to 0.6	SILT [TOPSOIL/FILL]	Soft [S]
0.6 to 2.2	Sandy SILT	Firm [F]
2.2 to 4.2	Silty SAND	Loose [L]
4.2 to 6.8	SAND	Medium Dense [MD]
6.8 to 8.0	Sandy GRAVEL	Medium Dense [MD]
8.0 to 9.5	SAND	Medium Dense [MD]
9.5 to 11.0	Sandy GRAVEL	Very Dense [VD]
11.0 to 20.0	SAND	Dense [D]

For the purpose of seismic foundation design, and on the basis of the site investigation and soil behaviour response, the soil classification in accordance with NZS 1170.5:2004 can be assumed to be "Class D – Deep or Soft Soil" (Standards New Zealand, 2004).

To provide a comparison to the absolute soil profile observed in the hand auger boreholes, test pits and machine boreholes, the SBTs from the CPTs conducted as part of the site investigation are summarised below in Figure 3-6.

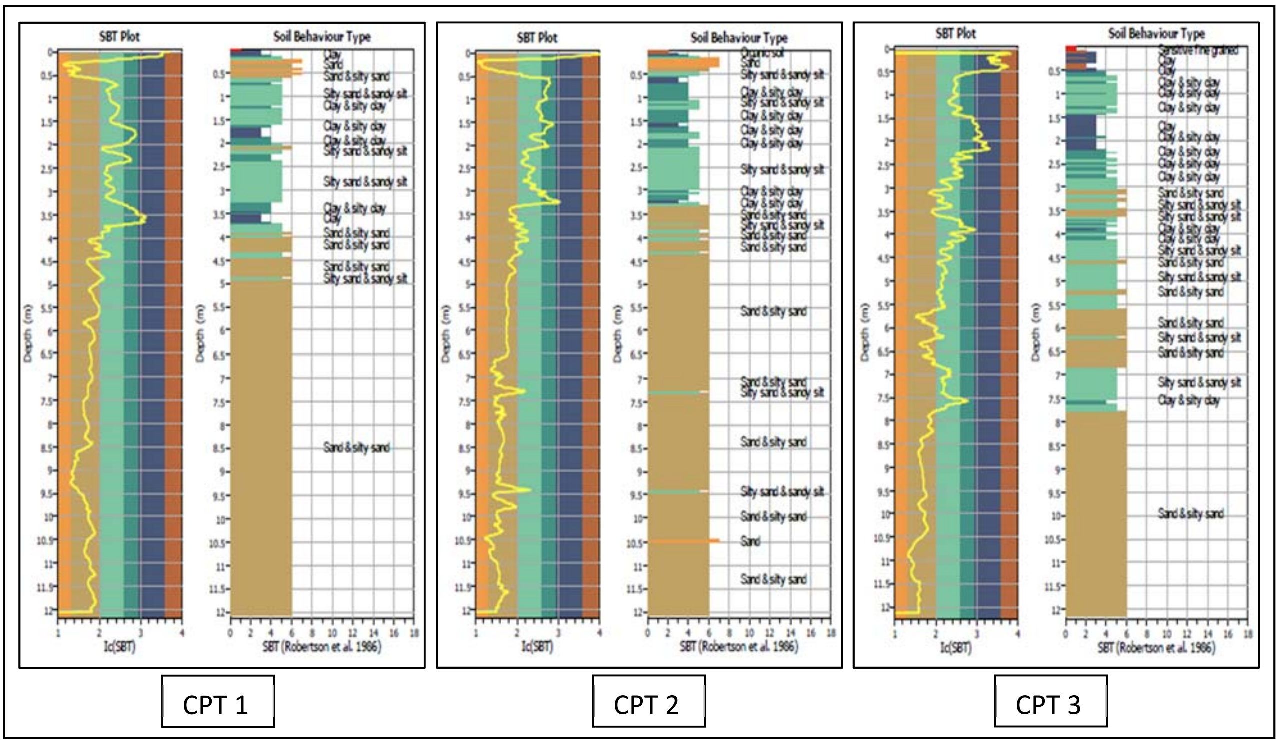


Figure 3-6 – Soil Behaviour Type (derived from CPTs)

The SBT results are somewhat consistent to the soil profiles defined by the hand auger boreholes, test pits, and machine boreholes. Interbedded silts and sands are observed to approximately 4.2 m bgl in the test pits and boreholes, while the SBT portrays interbedded silts and sands within the upper 4.5 m to 5.5 m bgl. Below 4.5 m to 5.5 m in the geotechnical investigation, the testing portrayed both a SBT and absolute soil profile of sand with localised lenses of gravel and silt.

A cross section was developed to using the test pit and borehole data to portray the approximate soil profiles across the site in Figure 3-7 below.

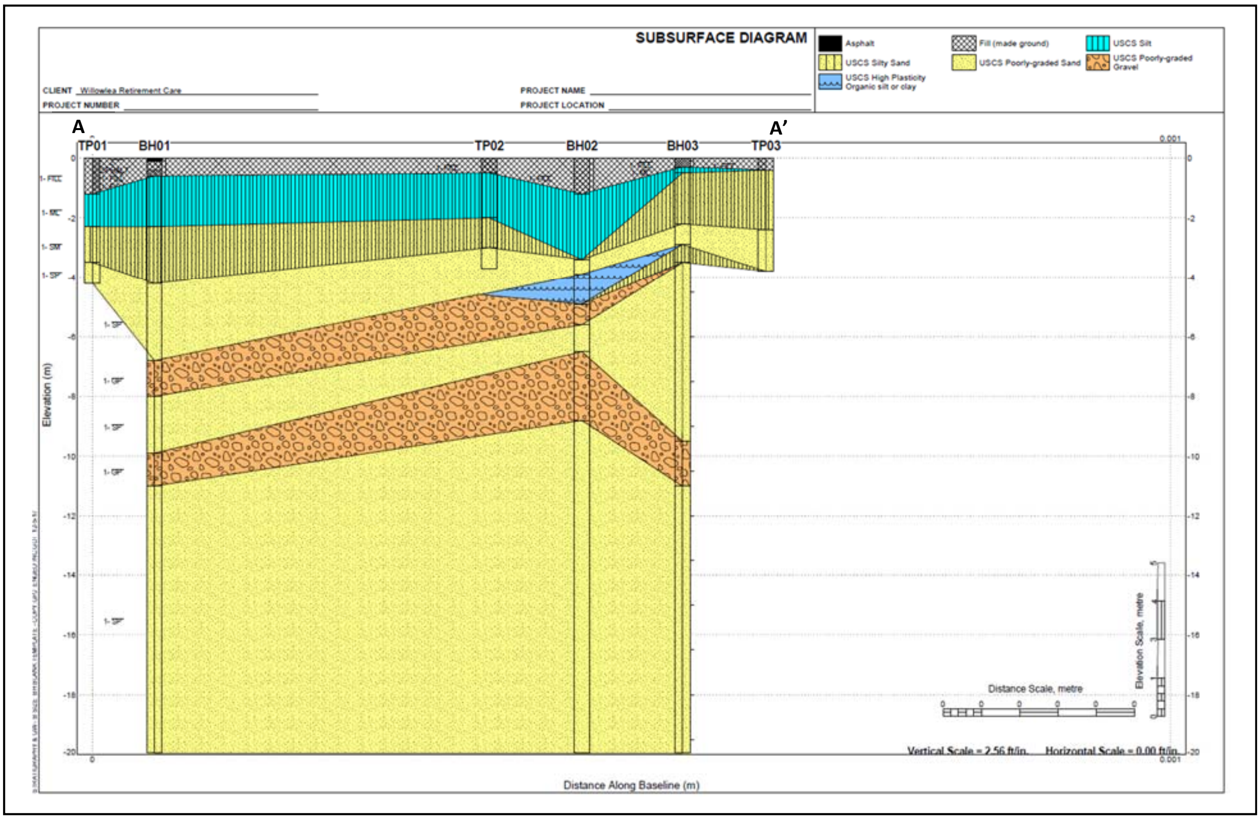


Figure 3-7 - Ground Model of Research Site

3.8 Conclusions

From the geotechnical investigation, which was completed using standardised investigation methods, the site portrays relatively consistent alluvial soils within the upper 20 m. The upper 3.5 m to 4 m below ground level are loose and soft interbedded sands and silts and become sandy and gravelly below 4 m.

The site investigation indicated in the test (BH02) on the inside of the Hills Road Drain (waterway adjacent to the site) had some more organic and clayey soils observed between 4 m and 4.5 m bgl. These were confirmed in CPT 3 which recorded similar organic and silty soils to approximately 4.5 m bgl.

Generally, the test methods conducted for the site investigation have recorded similar geology typical of Christchurch alluvial soils previously mapped by Brown & Weeber, (1992) and Forsyth, (2008).

4 *In Situ* Test Methods

4.1 Introduction

The objective of this research is to assess the ground improvement and densification of *in situ* soils around DSM columns within alluvial soils typical to Christchurch. Existing DSM test methods, focus on the soilcrete column specifically. However, as DSM is a ground improvement technique, an area identified for further research involves a focus on *in situ* field tests being conducted in soils around DSM columns to quantify the soil characteristic changes prior to and following DSM ground improvement. For this research, investigation of *in situ* soils around DSM columns was conducted through the implementation of standardised field testing methods. The test methods used are the CPT, DMT and sDMT tests which are standardised by the ASTM and are described in further detail in subsequent sections of this chapter.

The general soil properties of the research site were investigated through soil classification tests and site-specific geotechnical investigation, which contributes to the general soil knowledge and profile of the site. *In situ* testing methods (CPT, DMT and sDMT) have been employed to acquire specific soil parameters which aid in the quantification of the characteristic changes of the soil following ground improvement. These *in situ* field methods were performed before and after DSM column installation to examine the changes in strength, stiffness and stress state within soil improved by DSM.

4.2 Site Summary

The geology at the research site, as described in Chapter 3, is mapped by Brown & Weeber, (1992) and Forsyth, (2008) and as “river alluvium, comprising gravel, sand and silt in active flood plains (Q1a) or grey river alluvium beneath plains or low level terraces (Q1a)”. As summarised earlier, the site investigation confirmed the previously mapped site specific geology as alluvial silts and sands derived from the Yaldhurst member of the Springston Formation.

In order to answer the research objective of how alluvial soils of Christchurch respond to DSM, a series of invasive test methods were selected and the methodology of each test is outlined in the forthcoming sections.

4.3 Outline of Field Testing

This research aims to examine the effect DSM columns have on the *in situ* soils (around the columns) to aid in characterising the changes in stiffness, strength and soil stress state.

To quantify changes to *in situ* soil characteristics, and to provide a comparison between the *in situ* soils prior to and following the installation of DSM columns, a specific testing regime was developed. The testing regime called for specific *in situ* soil testing (CPT, DMT and sDMT) to be undertaken prior to the installation of DSM columns across three specific locations at the research site (Figure 4-1). These tests were completed to provide a baseline or standard of the existing ground response. To measure the response or change in soil characteristics, the final round of testing was completed in the same locations to provide a direct comparison following DSM column installation.

Minimising the potential disturbance within the test locations is required to mitigate possible interference to the results during testing. The *in situ* tests (CPT, DMT and sDMT) were completed at 500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm intervals from the column centre. Testing was completed approximately 3 days before column installation and a minimum of 28 days after column installation. In order to reduce the potential for disturbed ground and skewed results, a minimum spacing of 0.5 m between test locations was established (see Figure 4-2 for test panel layout). Previous research suggests that a minimum distance of between 0.5 m to 1 m between nearby invasive tests is appropriate (Marchetti, et al., 2001). The tests were completed at different proximities to the specific column locations. Varying the testing spacing around the columns enabled the quantification of the lateral confinement and measurement of the transition zones (if any) within the *in situ* soil.

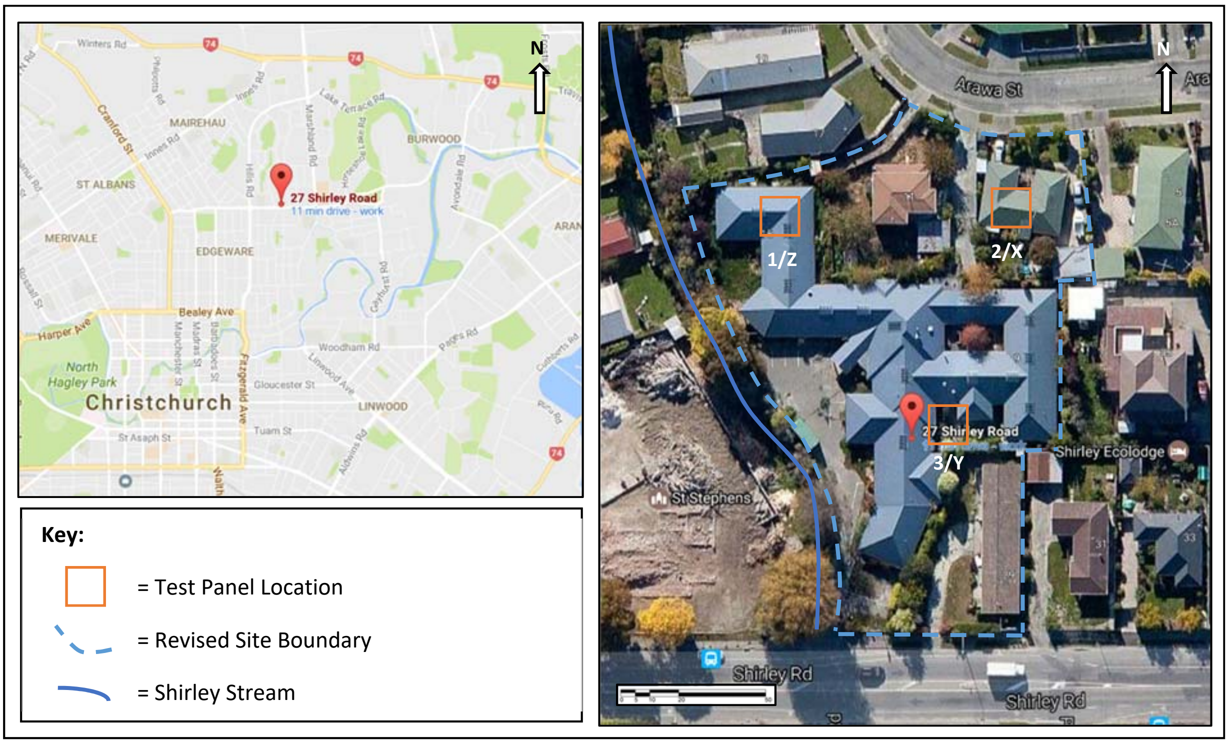


Figure 4-1 – Geotechnical Testing Locations (Google Maps, 2017)

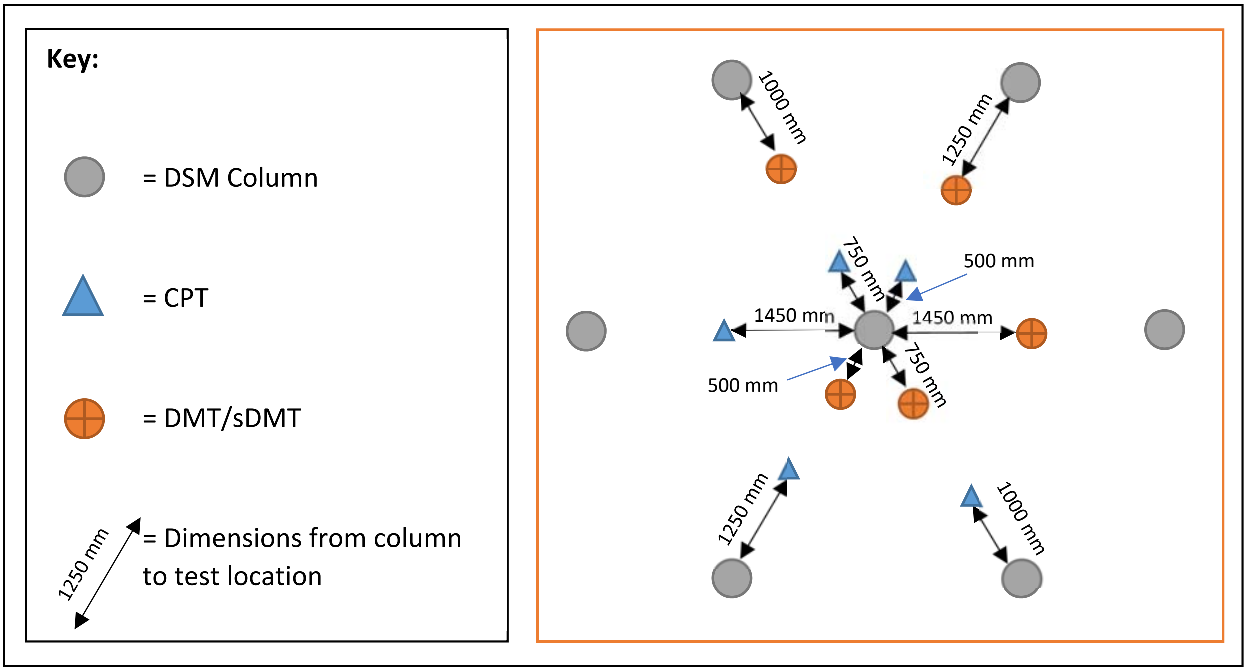


Figure 4-2 – Testing Panel with Layout of Test Locations Relative to DSM Column

4.3.1 Cone Penetrometer Test (CPT) Apparatus and Procedure

The CPT testing apparatus consists of an instrumented cone (with a usual apex of approximately 60°) facing down (Figure 4-3), load cell, friction sleeve, a series of hollow 1 m long rods and a machine capable of producing a constant hydraulic push for the hydraulic cone. The CPT test procedure uses either a light or a heavy rig (which is application dependant) to push the cone at a constant rate of 10-30m/s.

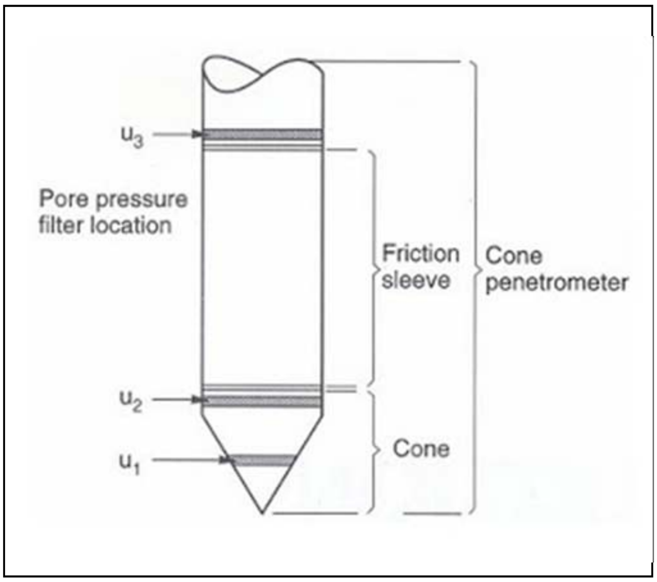


Figure 4-3 - Cone Penetrometer Apparatus (Lunne, et al., 1997)

As previously mentioned, testing was completed prior to and following DSM installation. For this research the initial testing was conducted using a Pagani TG63-150 1.5t hydraulic rig (Figure 4-4) where the rig is augured into the ground to provide resistance while the rod is advanced into the ground (see Appendix G for Rig Factsheet). During testing, auguring into the mixed fill and granular material following demolition proved to be slow and time consuming. To minimise the time spent on site, and to reduce costs for the CPT testing (completed following the installation of DSM columns), the heavier 6x6 Man Geomil rig (Figure 4-5), which has a 20 t push capacity, was used (see Appendix G for Rig Fact Sheet).

As the cone is advanced into the ground, the resistance to the cone is recorded (q_c). As the cone advances further, the sleeve and cone move together creating the sleeve resistance (q_t). This is continued until the desired test depth is reached by adding additional rods as required. For this research, CPT testing was conducted to 12 m bgl with an advancement of 20 mm/s, and with a ground record every 10 mm.

The testing standard complied with for this research was the "ASTM D5778-12 Standard Test Method for Electronic Friction Cone and Piezocone" (ASTM, 2012). Refer to Appendix A for further information.

The method for CPT analysis adopted for the Canterbury Rebuild (and for this research) is described by Boulanger & Idriss, (2014). This method has been been refined using testing derived from the Canterbury Rebuild for calibration. Although this method can tend to overestimate settlements in some soils (Van T Veen, 2015), currently, this is the most accurate method for analysing soil response due to calibration from test conducted specifically for the Canterbury Rebuild process.

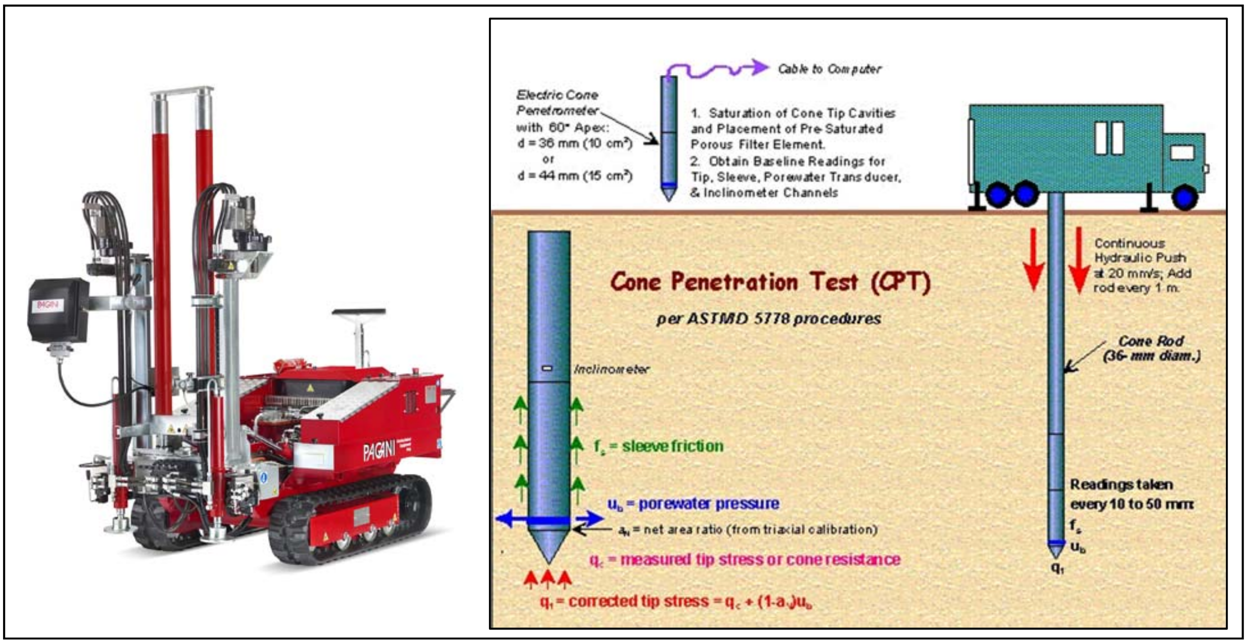


Figure 4-4 - Cone Penetrometer Testing Apparatus and Procedure (Pagani Geotechnical Equipment, 2017)

Figure 4-5 - Site Photographs (CPT testing post DSM Installation)

4.3.2 Flat Dilatometer Test (DMT) Apparatus and Procedure

The DMT test apparatus included a flat blade with a flexible, circular steel membrane mounted flush on one side. The blade is connected to a control unit situated on the ground surface via a pneumatic-electrical tube (Figure 4-6) (which transmits gas pressure and electrical continuity) (Marchetti, et al., 2001).

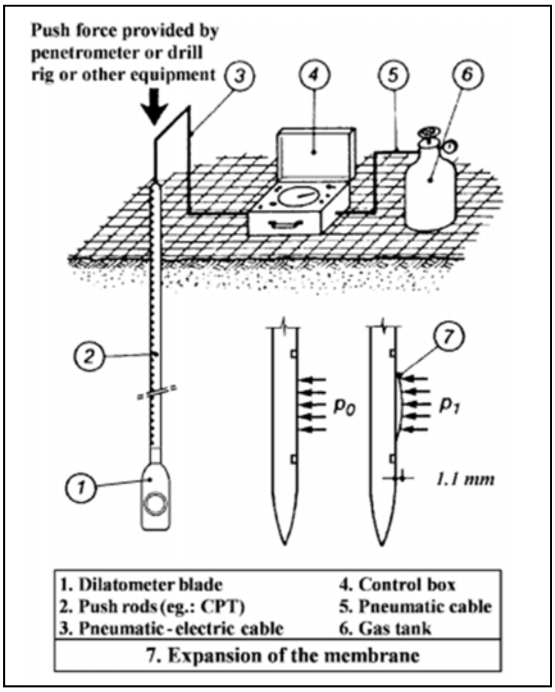


Figure 4-6 - Flat Dilatometer (DMT) (Marchetti, et al., 2001)

As previously mentioned, DMT testing was completed prior to and following DSM installation to form a comparison of the soil characteristic response to ground improvement. The DMT test is a standardised method which attaches to a hydraulic CPT rig. Therefore, for this research the initial and post DSM testing was conducted using a Pagani TG63-150 1.5t hydraulic rig (Figure 4-4). Like the the CPT testing procedure mentioned above, the rig was augured into the ground to provide resistance as the blade is advanced into the ground (see Appendix G for Rig Factsheet). During testing, auguring into the mixed fill and granular material following demolition proved to be slow and time consuming. However, this testing method was unable to be revised due to a shortage of larger push rigs at the time of this research.

The testing procedure was performed by advancing the DMT blade into the ground and stopping at 0.2 m intervals. At each 0.2 m interval an inflation of the membrane occurs. The initial reading (A-pressure) is made and records how much pressure is required to expand the membrane against the *in situ* soil. Subsequently, the second pressure reading (B-pressure) is taken and records how much pressure is required to advance the centre of the flexible membrane 1.1 mm against the *in situ* soil. An optional reading (C-pressure) can be conducted, which measures the closing pressure after the B pressure reading is recorded. For this research, the A and B pressure readings were completed at 0.2 m intervals, however the C-Pressure was not recorded.

Figure 4-7 - Site Photograph (DMT Testing)

The testing standard complied with for this research was the "ASTM D6635-15 Standard Test Method for Performing the Flat Plate Dilatometer" (ASTM, 2015). Refer to Appendix A for further information.

The method for DMT analysis adopted for this research is described in the paper written by Marchetti (2014). Currently, this is the most accurate method for analysing soil response to the DMT.

4.3.3 Seismic Flat Dilatometer Test (sDMT) Apparatus and Procedure

The sDMT is an adaptation to the DMT method where a cylindrical seismic module is mounted above the DMT blade with two seismic geophones included at a spacing of 0.5 m.

As the DMT is advanced, the operator stops the blade at 0.5 m intervals within the *in situ* soil. The shear wave source at the surface is a 10 kg pendulum hammer. This is released and strikes a steel plate orientated long axis parallel to the receivers to generate the highest sensitivity wave. The plate is wedged against the soil by the base of the operator's truck. As the pendulum hammer strikes the steel plate, the waves propagate through the *in situ* soil towards the two receivers in the seismic module. The seismic module records the zero time signal as the hammer impacts the steel plate and subsequently records how long it takes wave to travel through the soil and pass the two receivers (Marchetti, et al., 2001). The wave responses in receiver 1 (S1) and receiver 2 (S2) are re phased together over the difference in time to produce a shear wave velocity reading.

As previously mentioned, testing was completed prior to and following DSM installation. For this research the initial testing was conducted using a Pagani TG63-150 1.5t hydraulic rig (Figure 4-4) where, like the CPT testing procedure mentioned above, the rig is augured into the ground to provide resistance as the blade and seismic module is advanced into the ground (see Appendix G for Rig Factsheet).

Figure 4-8 - Site Photograph (sDMT Testing)

The testing standard complied with for this research was the "ASTM D6635-15 Standard Test Method for Performing the Flat Plate Dilatometer" (ASTM, 2015). Refer to Appendix A for further information.

The method for sDMT analysis adopted for this research is described by Marchetti (2014). Currently, this is the most accurate method for analysing soil response to the DMT.

4.4 Aims of Testing and Synthesis

Although there has been some research conducted using CPT, DMT and sDMT testing in clayey soils, there has been limited study focusing on the soil changes around DSM columns in an alluvial soil context. Comparatively, there has been extensive *in situ* and laboratory testing which focuses on the soilcrete within DSM columns (not the *in situ* soils around them). Larsson (2005) provides a thorough overview of these analyses.

The objective of this research is to assess the ground improvement of (untreated) soils around DSM columns within alluvial soils of Christchurch. The research aims to minimise the focus of existing quality assurance testing on DSM columns themselves by determining whether the desired increases in *in situ* soil stiffness can be measured using simple, repeatable, cost effective and efficient test methods.

By focusing on the *in situ* soil characteristic changes, testing methods used can be less invasive and potentially damaging to the DSM columns and provide a more cost effective testing platform for design. Testing using CPT, DMT and sDMT can aid in quantifying soil stiffness, strength, and stress state of the *in situ* soils around DSM columns, the primary objective of this research. The results of the proposed testing are summarised in Chapter 5.

5 Results

5.1 Testing Summary

In order to begin to answer the research objectives of this thesis, a series of standardised invasive test methods were selected. The main objective is to assess the ground improvement of ("untreated") soils around DSM columns within alluvial soils typical to Christchurch.

In situ testing was completed at the research site, in the overbank deposits of the Springston Formation within the upper 12 m. The soil previously investigated at the site is classified as interbedded sandy silt and silty sand up to approximately 4.5 m bgl. From 4.5 m to 20 m bgl, medium dense to dense sandy gravel and sand was observed.

The columns were installed in an open grid layout with a 0.9 m diameter and were spaced 2.9 m apart, centre to centre. CPT, DMT and sDMT tests conducted between installed DSM columns enabled the examination of soil characteristic changes within the soils around DSM columns following ground improvement. The *in situ* tests were completed at 500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm from the column centres. To minimise ground disturbance, the tests were spaced in a predetermined layout to establish a minimum spacing of 0.5 m.

CPT, DMT and sDMT testing was competed before and after DSM column installation to provide a direct comparison of the soil response to DSM. Three locations across the site were selected for this research (see Figure 4-1 in Chapter 4), where preliminary testing was completed (to provide an untreated soil response) approximately 3 days before column installation and post construction testing was conducted a minimum of 28 days after column installation (to provide an 'improved' soil response).

The results of the *in situ* testing is outlined in this chapter. Further interpretation of the results is discussed in Chapter 6 and Chapter 7.

The data collected was derived using empirical methods, which estimate the behavioural response the soil has to the *in situ* tests, and thus enables comparisons to be made between each test method. However, due to the empirical nature of the methods, some limitations of the results need to be considered. These potential limitations are discussed in the conclusions of this chapter and the limitations section in Chapter 7 of this thesis.

Prior to this research there have been previous investigations which focus on the response *in situ* soils have to various methods of ground improvement. Guetif, et al., (2007) have numerically modelled the effects of stone column installation (and ground improvement) on *in situ* soils by quantifying increased soil stiffness before and after column installation. Similarly, Chai, et al., (2005) and Chai, et al., (2007) have both examined the response of the soil characteristic changes to *in situ* clayey soils around DSM columns. Tatarniuk (2014) has further investigated the *in situ* soil response around DSM in a clayey soil context.

All these investigations have predominantly used empirical invasive test methods for soil response analysis and have suggested positive correlations between ground improvement methods and *in situ* soil response. This is why invasive test methods, which have been previously used with success, were been selected for this research.

5.2 Liquefaction Potential

At the research site, 3 CPT tests were performed across three different locations before the installation of DSM columns, and 15 CPT tests were carried out (in the same three locations) after DSM column installation. These tests were terminated at a target depth of approximately 12 m bgl to provide an understanding of the soil profile within the DSM zone (8.5 m bgl) and the unmixed soil profile below.

Similar to the site investigation completed prior to this research, the SBT of the *in situ* soils encountered was portrayed by the CPT as interbedded silts and sands within the upper 4 m bgl, followed by sands and sandy gravels to 12 m bgl (Figure 5-4). CPT testing was used to measure subsurface stratigraphy and the geotechnical parameters of the site specific soils. Within the calibrated steel cone, cone resistance (q_c), sleeve friction (f_s), and pore pressure (u) were measured as the module was advanced into the ground at a constant rate and empirical calculations were completed for further analysis of the soil profile.

Using CLiq liquefaction software (Geologismiki, 2017), the liquefaction potential of the *in situ* soils preceding and following DSM column installation were examined using CPT inputs from the procedure outlined by Boulanger & Idriss (2014).

Following the CES, Christchurch adopted a standard limit state design (LSD) method for residential redevelopment. This was developed to fulfil two specific criteria: serviceability limit state (SLS) and ultimate limit state (ULS). SLS design is defined as the ability of a structure to respond to an earthquake event and still remain useable as the original design intended. By contrast, ULS design is defined as the ability of a structure to prevent collapse, remain standing and enable occupants to evacuate, but may require significant repair or rebuilding following a seismic event (Van T Veen, 2015).

As recommended by MBIE for Class D sites (deep and/or soft soil sites) in Christchurch, the SLS case 1 event was modelled using a 0.13g, M7.5 earthquake and the SLS case 2 event was modelled using a 0.19g M6.0 earthquake. The ULS event was modelled using a 0.35g M7.5 earthquake.

The liquefaction potential of the ULS, SLS case 1 and SLS case 2 parameters from location Y (Figure 5-1, 5-2 & 5-3) (with the remainder of the test results plotted in Appendix J), are plotted below to 12 m bgl.

The outputs of the liquefaction analysis are split into 5 outputs: the cyclic resistance ratio (CRR), the factor of safety, the liquefaction potential index (LPI), the predicted vertical settlement and the predicted lateral (or horizontal) settlement.

The CRR an empirical method used to measure the soil's resistance to liquefaction during an earthquake event. The method is a simplified shear stress equation which is a function of the earthquake shaking duration, soil relative density, effective confining pressure, non-plastic fines content, seismic harmonic cycles and some other soil properties (Jafarian, et al., 2013).

The factor of safety (FOS) represents the susceptibility the soil has to liquefaction as a function of the soil's capacity to resist liquefaction (CRR) to the seismic loading applied to the soil element using the response from *in situ* tests (Lo Presti & Meisina, 2014). Simply, if the FOS is less than 1, the soil will likely liquefy in the event modelled. Conversely, if the FOS is above 1, the soil is unlikely to liquefy.

The LPI considers the soil thickness, depth of test and FOS value of the soil layers (Lo Presti & Meisina, 2014). The plot is split into green, orange and red sections. The green plot suggests low liquefaction potential, the orange plot suggests moderate liquefaction potential, and the red plot suggests likely liquefaction potential as a function of the modelled earthquake event.

Predicted vertical settlement suggests how much predicted settlement is likely to occur vertically as a function of the soil type, soil response and modelled earthquake event. This is plotted as a function of depth and the predicted settlement in millimetres and centimetres. The method used to calculate predicted settlements from CPT and SBT outputs recommended for the Canterbury rebuild by MBIE, is the method described by Boulanger & Idriss (2014).

The predicted horizontal settlement suggests how much settlement is likely to occur horizontally as a function of the site, proximity to a free face (waterway), slope of the site, soil type, soil response and modelled earthquake event. This is plotted as a function of depth and the predicted settlement in millimetres and centimetres. The method used to calculate predicted settlements from CPT and SBT outputs recommended for the Canterbury rebuild by MBIE, is the method described by Boulanger & Idriss (2014).

The plots in Figures 5-1, 5-2 & 5-3 include the preliminary test (blue) and the other post DSM tests. Figure 5-1 suggests the ULS response portrays decreased liquefaction potential in the post DSM tests from 750 mm to 1450 mm outside the column centre. The test conducted 500 mm from the column centre (indicated in yellow) suggests increased liquefaction potential or deterioration.

Figures 5-2 and 5-2 (SLS case 1 and SLS case 2) portray the LPI and FOS as suggesting low liquefaction potential due to the modelled seismic event. However, despite this, similar results to the ULS plot were observed where the test conducted 500 mm from the column centre has an increased potential for liquefaction compared to the pre DSM test.

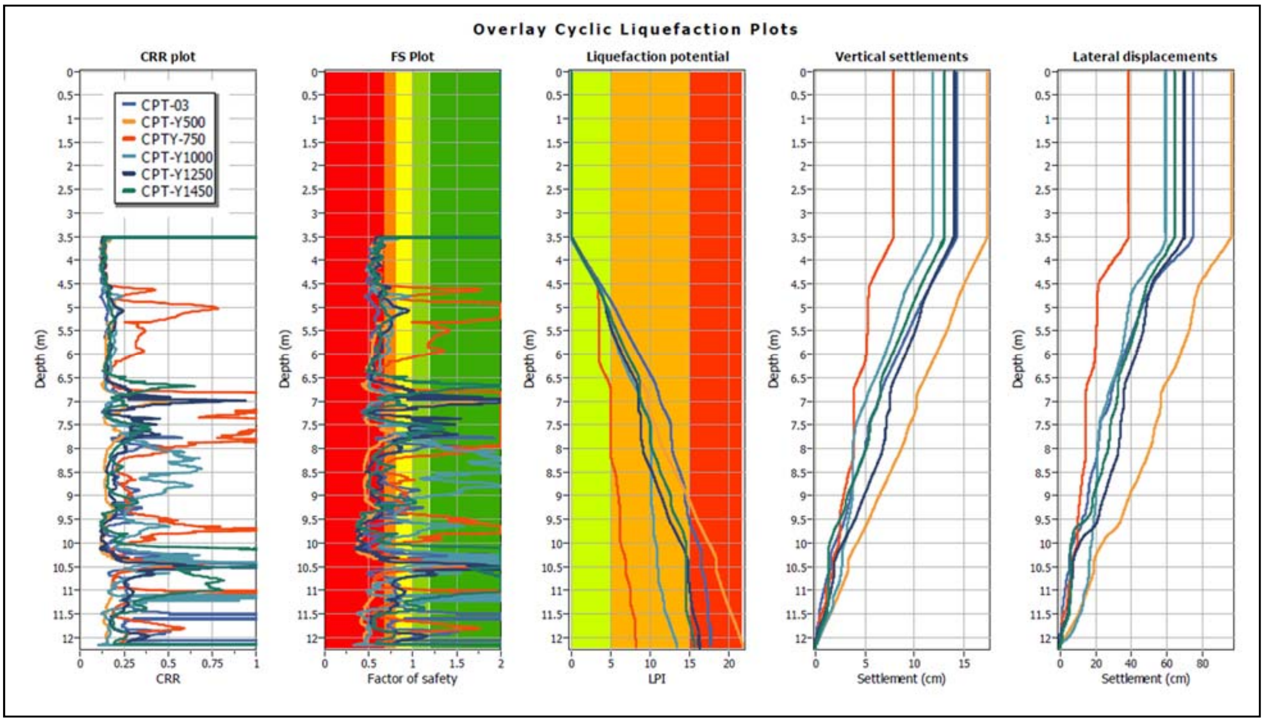


Figure 5-1 – Predicted Liquefaction Potential Plots ULS exported from CLiq at Test Location Y

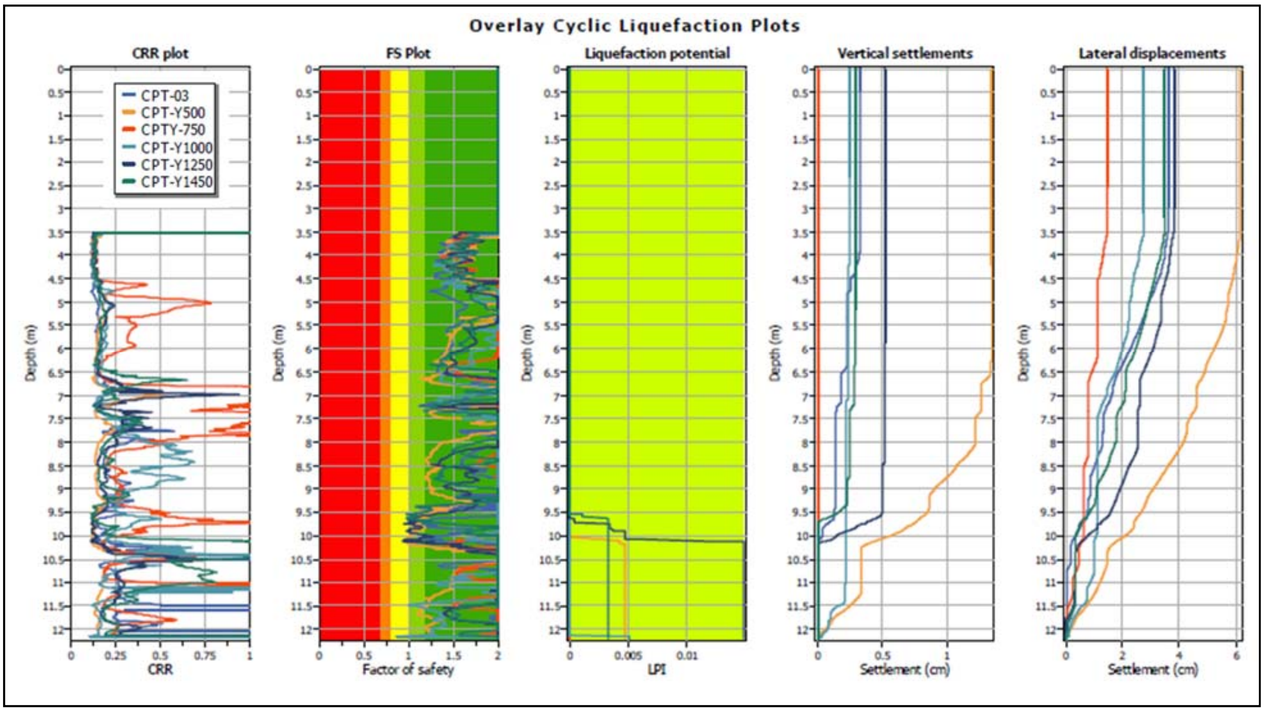


Figure 5-2 – Predicted Liquefaction Potential Plots SLS Case 1 exported from CLiq at Test Location Y

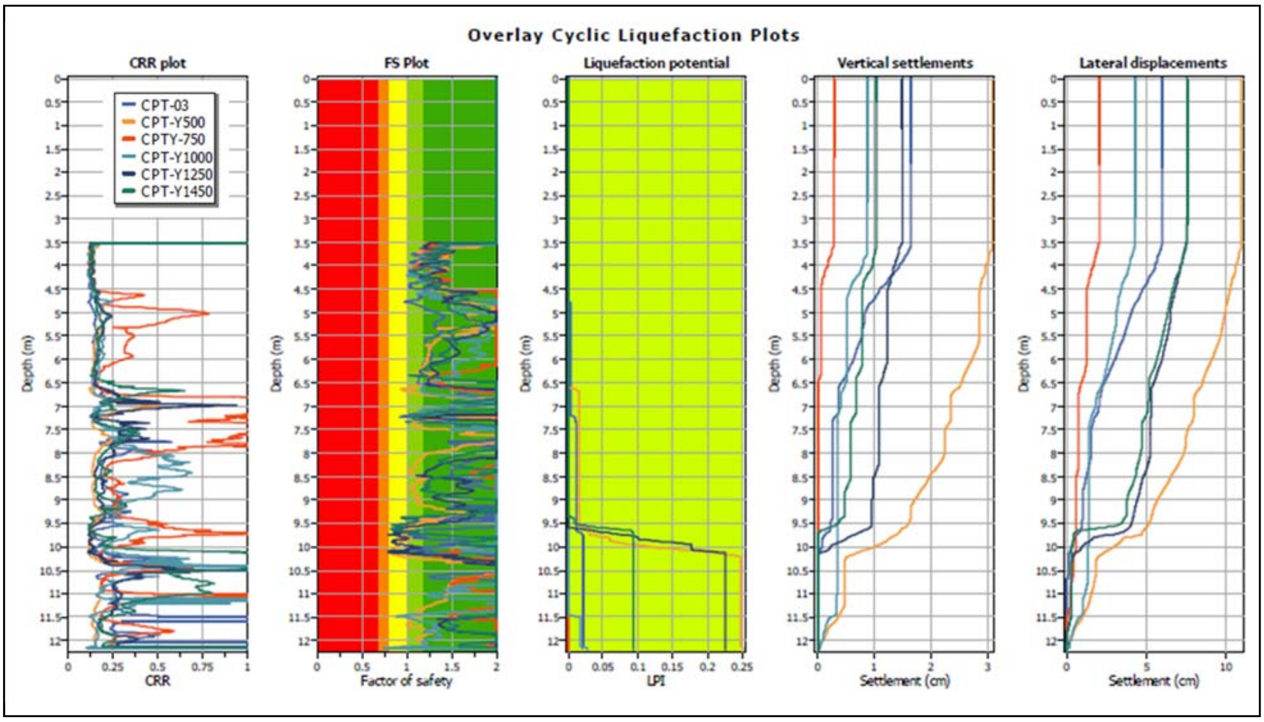


Figure 5-3 – Predicted Liquefaction Potential Plots SLS Case 2 exported from CLiq at Test Location Y

5.3 Material Index (I_D)

To empirically estimate the *in situ* soil type or mechanical behaviour of the soils and define the material index (I_D) of *in situ* soils, the DMT was performed at the research site. Marchetti (1980) assumes the typical soil boundaries when conducting the DMT test are classified in accordance with Table 4.

Table 3 - Soil Classification using I_D and the DMT (Marchetti, 1980).

Soil Type	Clay		Silt			Sand	
	Clay	Silty Clay	Clayey Silt	Silt	Sandy Silt	Silty Sand	Sand
I_D Value	0.1	0.35	0.6	0.9	1.2	1.8	3.3

I_D provides a useful index for the identification of different soil compositions (typically sand, silt and clay). I_D does not interpret soils as a function of absolute soil particle size (by completing a sieve analysis or equivalent method), rather it describes the empirical behavioural response of a soil. This output can be compared to the CPT SBT to provide assurance of the soil profile behaviour.

At the research site, DMT testing was carried out prior to construction of DSM columns in three predetermined locations (X, Y and Z). At these same locations, following DSM column installation, five tests were performed (totalling 15 tests) to a depth of approximately 12 m bgl. The method of conducting I_D is summarised in detail in Chapter 2. Equation 2.8 (as repeated below), which incorporates the corrected first and second pressure readings and the pre-insertion pore pressure, defines the empirical soil response derived from the DMT procedure.

$$I_D = \frac{p_1 - p_0}{p_1 - u_0} \tag{2.8}$$

The results from location Y (all test results plotted in Appendix L) are plotted below to 9 m bgl. This allowed an analysis of the ground improvement effects as the DSM columns were installed to approximately 8.5 m bgl. The results from the CPT SBT index (I_C) are shown adjacent to the I_D plots to provide a comparison.

The I_D plots demonstrate the behaviour of the soil in response to the DMT with plots of the soil classification index values derived from Table 4. The soils are plotted between the index value of 0.9 (silt) to 3.3 (sand). When compared to the CPT SBT (I_C), the soil derived from the DMT (I_D) portrays a similar response.

The results of I_D show a general increase in soil stiffness as a function of DSM column installation. The soils plotted around the silty sand to sand cut off value tended to increase, with the highest increase recorded in the test completed 500 mm from the column centre. As the tests were completed further from the DSM columns, the less increase in soil stiffness was recorded. An average increase in soil stiffness over the improved soil depth (8.5 m bgl) was approximately 20%.

The silty soils within the upper 3.5 m portrayed a less consistent increase in stiffness as a result of DSM compared with the granular soils observed between 3.5 to 8.5 m bgl. The clayey soils at around 3.2 m bgl portrayed no change in I_D before and after DSM column installation.

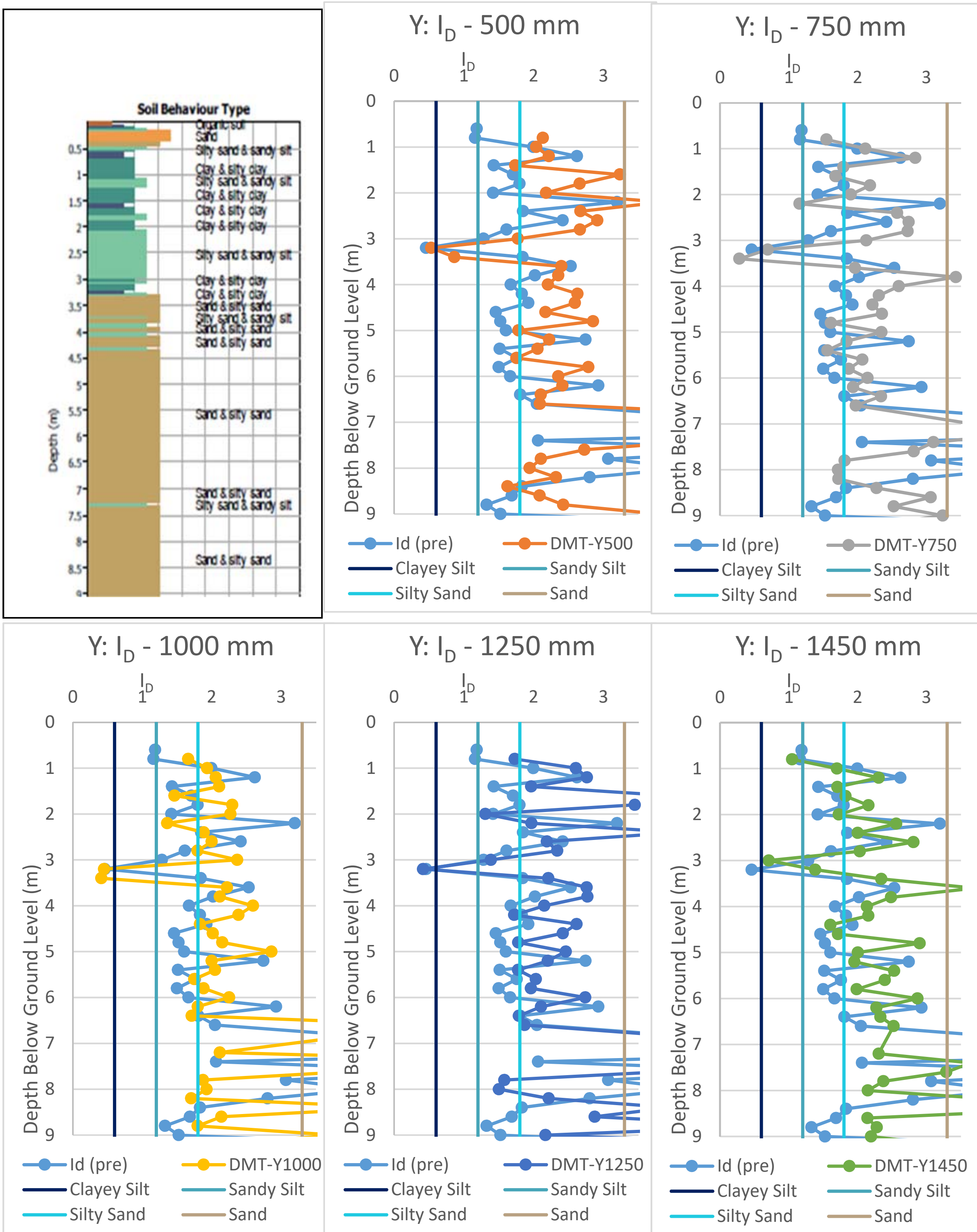


Figure 5-4 – Material index (I_D) versus depth before and after DSM installation with soil cut off values.

5.4 Horizontal Stress Index (K_D)

The horizontal stress index (K_D) is an important parameter when characterising the soil behaviour and response to dynamic loading of *in situ* soils.

K_D is a parameter empirically derived from the DMT test which estimates the *in situ* soil's resistance to liquefaction or the soils resistance to volumetric change during seismic events (Monaco & Marchetti, 2007). This parameter is a function of the pressure of the soil acting horizontally on the DMT blade or the resistance of the soil to lateral movement during testing within the *in situ* soil profile. Therefore, as a general rule, the higher the K_D value, the greater the soil stiffness and the less likely the soil is to liquefy during a seismic event.

K_D is summarised in Chapter 2, and defined by Equation 2.9 (repeated below), where the equation includes the empirical pressure response of the soil, with the pre-insertion hydrostatic pore pressure and the pre-insertion overburden stress.

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \tag{2.9}$$

As previously summarised in section 5.3.4 above, at the research site, DMT testing was carried out prior to and following construction of DSM columns in three predetermined locations to a maximum depth of 12 m bgl. The results from location Y are plotted below (Figure 5-5) with the remainder of the test results shown in Appendix L. The results from the SBT I_c index (derived from the CPT) are presented next to the plots to provide a comparison of soil type with depth.

The K_D plots portray an increase in soil response to DSM column installation within the granular soils as a function of the lateral stress applied to the *in situ* soils from the DSM process. Across the treated soil profile 8.5 m bgl, there was a recorded increase in K_D of approximately 10%. The silty soils within the upper 3.5 m portrayed minimal change in soil stiffness following DSM compared with the granular soils observed between 3.5 to 8.5 m bgl. The results show a marked decrease in K_D 500 mm away (approximately 7%) from the column centre within the soil profile.

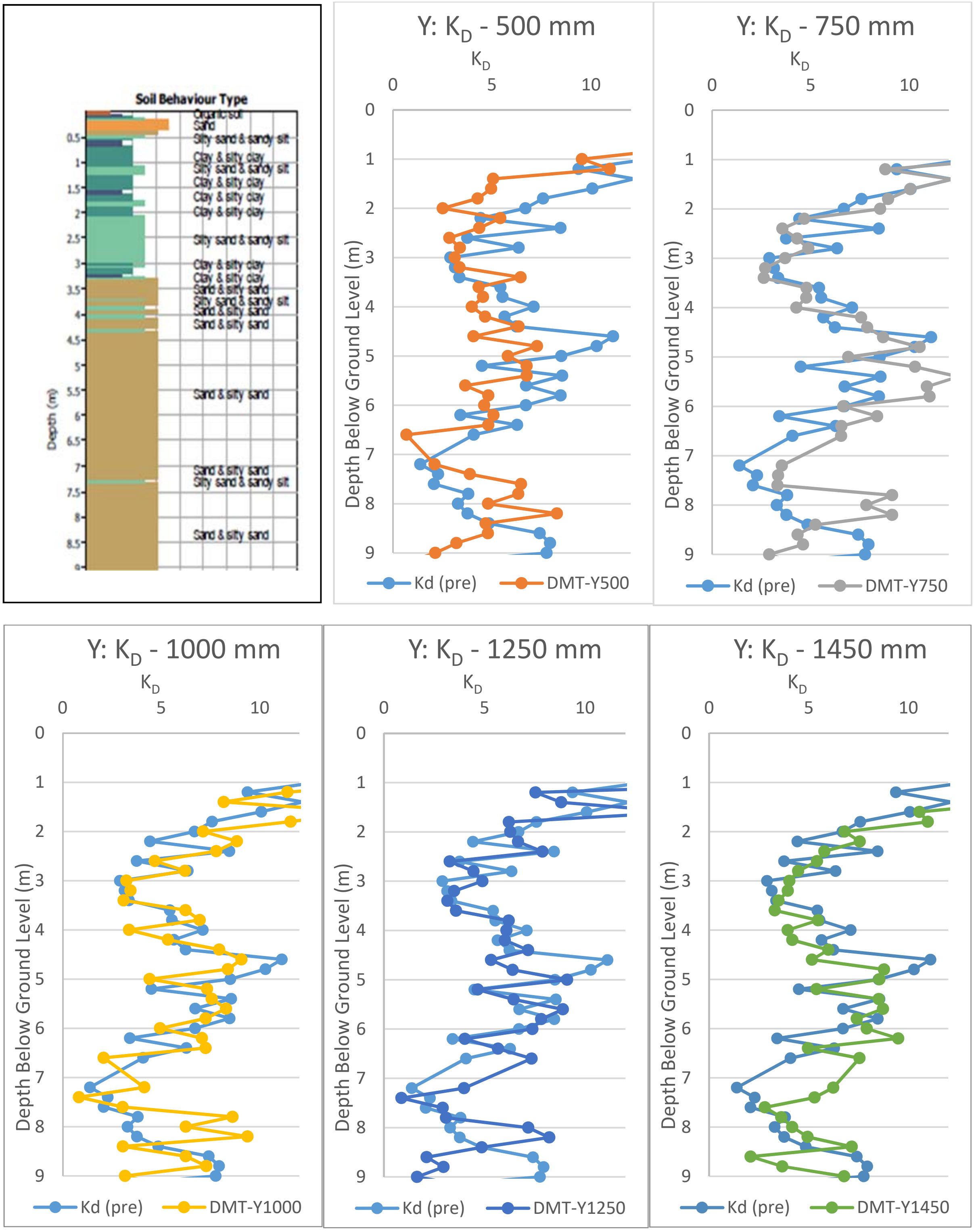


Figure 5-5 – Horizontal stress index (K_D) versus depth before and after DSM installation.

5.5 Shear Wave Velocity (V_s)

The shear wave velocity (V_s) is a key parameter for understanding the influence seismic waves have on *in situ* ground. V_s is derived in this research from the seismic flat dilatometer test (sDMT) which measures the response *in situ* soils have to a generated signal at the ground's surface.

The sDMT is an adaptation to the traditional "mechanical DMT" introduced and developed by Marchetti (1980) with a seismic module fitted above the DMT blade.

Since the sDMT is simply an adaptation of the DMT, the testing procedure was carried out in conjunction with the DMT tests. Therefore, like with the DMT testing process previously mentioned, sDMT tests were conducted in three predetermined locations (Locations X, Y and Z) (see Figure 4-1)

The shear wave testing was conducted in 0.5 m depth intervals as the seismic module was advanced into the ground. At these locations, following DSM column installation, five sDMT tests were performed (totalling 15 tests) to a depth of approximately 12 m bgl.

V_s methods are summarised in Chapter 4 and the previous research defined in Chapter 2, with the empirical Equation 2.12 (as repeated below) which derives V_s as the difference in the distance between each of the seismic geophone receivers divided by the arrival delay period from the initial signal impact.

$$V_s = (S_2 - S_1)/\Delta t \tag{2.12}$$

The results from location Y are plotted below to 9 m bgl to account for the ground improvement installation depth of 8.5 m bgl. The remainder of the test results are plotted in Appendix J. The results from the SBT index I_c are presented next to the plots to provide a comparison of soil type with depth.

Generally, the faster the V_S, the greater the density and stiffness recorded within the soil profile, and the higher the CRR is and thus the less prone the soils are to liquefy in a future seismic events.

The V_s plots portray a marked increase in all the tests conducted within the *in situ* soils after DSM column installation. Approximately 25% increase in soil stiffness was recorded across the entire soil profile (8.5 mbgl). The greatest increase is recorded in the granular soils in the test conducted 500 mm from the column centre with an average recorded increase of 55% in V_S. As the tests are conducted further from the column, the increase in V_S diminishes. Within the silty soils (upper 3.5 m) in the test conducted 500 mm from the column centre, the soils have portrayed a mixed response to DSM.

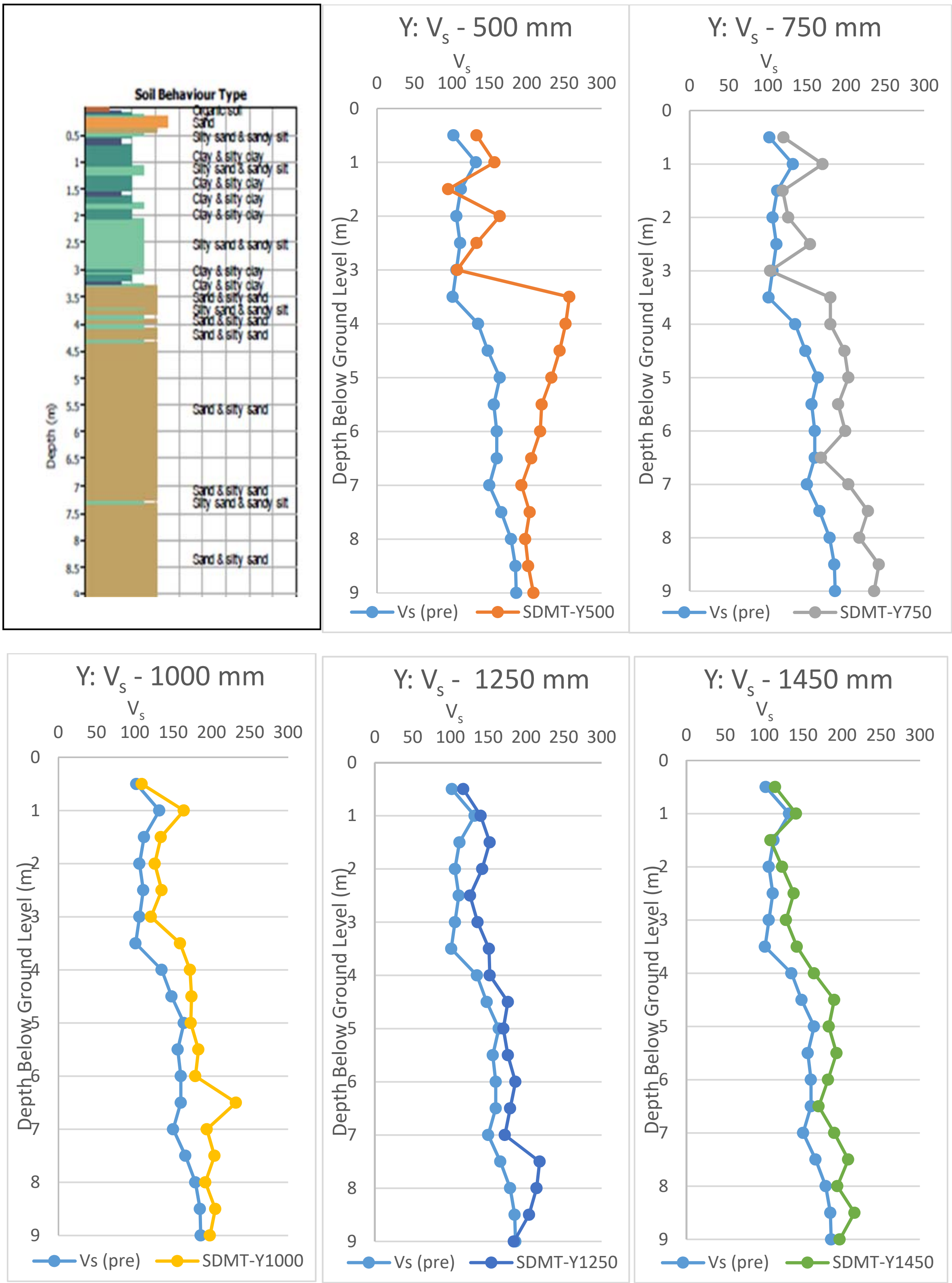


Figure 5-6 – Shear Wave Velocity (V_S) versus depth before and after DSM installation.

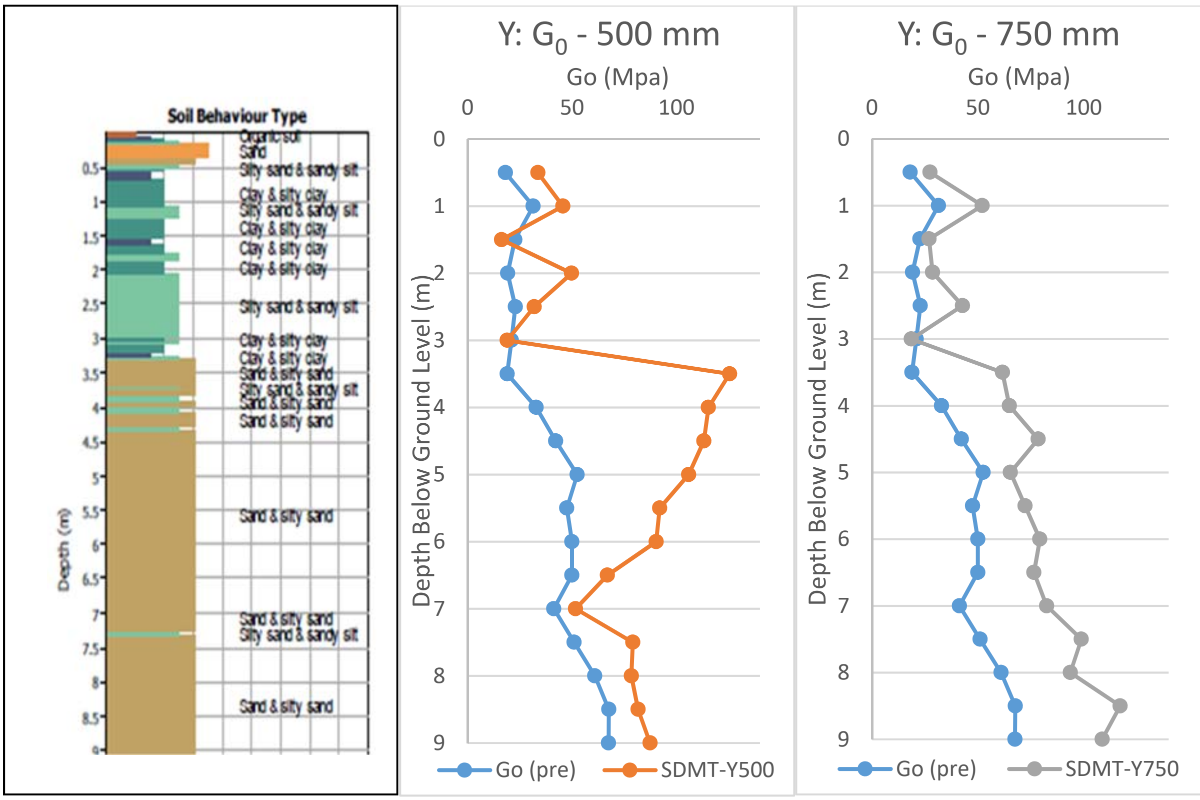
5.6 Small Strain Shear Modulus (G_0)

The small strain shear modulus (G_0) is a direct function of V_s during sDMT testing. G_0 is outlined in Chapter 2, and Equation 2.14 (as repeated below) summarises G_0 by incorporating V_s and the bulk density of the soil medium as the shear waves are propagating through. G_0 records the amount of strain developed when soils undergo loading.

$$G_0 = \rho V_s^2 \tag{2.14}$$

As previously summarised in above, sDMT testing was carried out before and after DSM column installation in three predetermined locations to a maximum depth of 12 m bgl. The results from location Y are plotted below, with the remainder of the test results in Appendix J. The results from the SBT index (I_c) are presented next to the plots to provide a soil comparison.

As the G_0 plots are a function of the V_S results, the G_0 graphs demonstrate an increase in all the tests conducted within the *in situ* soils after DSM column installation with an approximate increase in soil response of 60% across the improved depth (8.5 mbgl). The silty soils within the upper 3.5 m portrayed mixed results, and less consistent improvement compared to the granular soils below 3.5 m bgl. As the tests are conducted further from the column, the increase in G_0 diminishes.



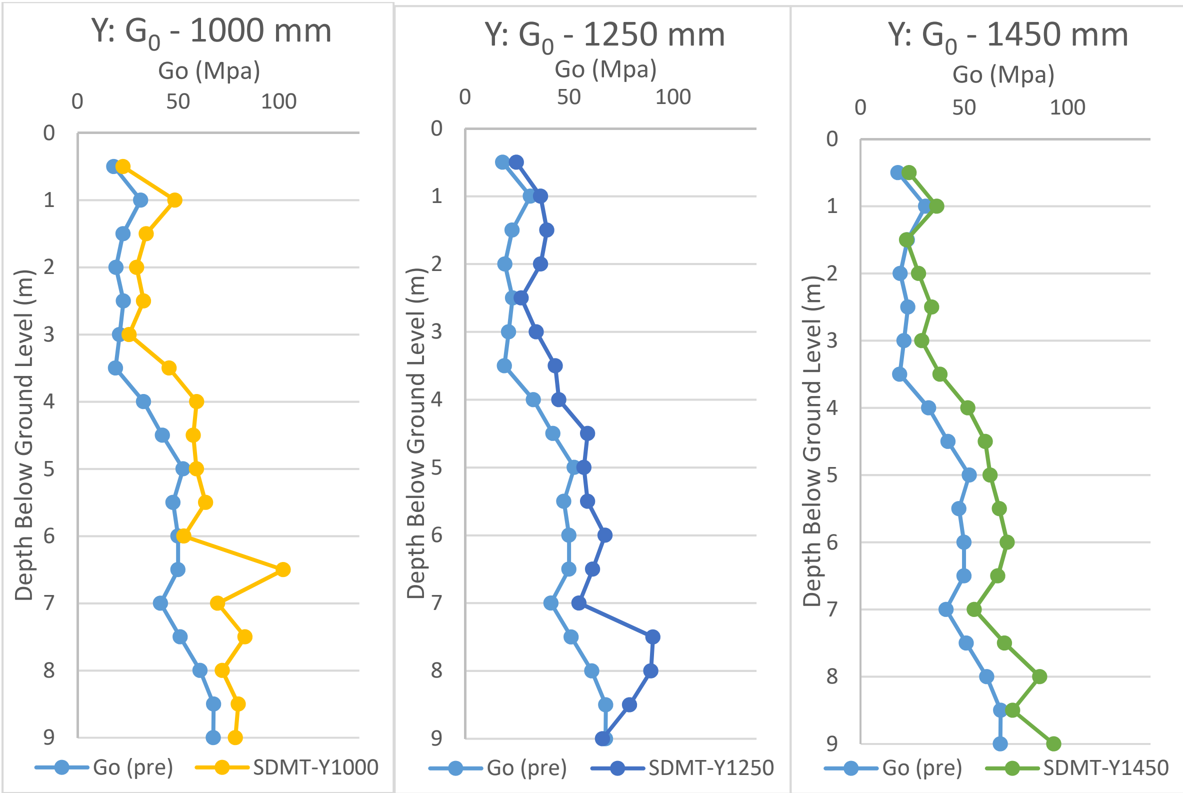


Figure 5-7 – Small Strain Shear Modulus (G_0) versus depth before and after DSM installation.

5.7 Conclusions

To measure the soil characteristic changes at the research site following the installation of DSM columns, testing was complete before DSM and after DSM installation. The CPT, DMT and sDMT tests were conducted in soils between installed DSM columns to establish the soil response to ground improvement. An average increase in *in situ* soil improvement was recorded and across the methods selected for this research *The* data collected from this research was empirical correlations of the soil response to DSM ground improvement. The results of *in situ* testing outlined in this chapter has been interpreted and discussed further in Chapter 6 of this thesis.

The justification for this testing arrangement was derived from current knowledge of the radial soil response within soils around DSM being poorly understood in a Christchurch alluvial soil context. The *in situ* tests (CPT, DMT and sDMT) were completed in three different locations across the site at different distances from the DSM column (500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm from the column centre) to establish whether the stresses applied to soils do in fact reduce as a result of the DSM column.

The empirical methods selected are standardised by ASTM and provide a valid platform for data analysis. The methods have proven to successfully record positive empirical soil responses. However, generally with any test method, there will always be limitations, and these tests completed for this research have some limitations worth considering which are discussed further in Chapter 7 of this thesis.

6 Discussion

6.1 Introduction

This chapter summarises and discusses the results portrayed in Chapter 5 in relation to the invasive testing conducted around the DSM columns at the research site before and after DSM column installation. Each of the test results are examined individually to provide a comparison between the different test methods and define conclusions of the research. Further discussion summarises how the results answered the objective of the research. The objective of this research is to assess the ground improvement of soils around DSM columns within alluvial soils in a Christchurch alluvial context.

Because DSM column installation involves a positive volume increase within the soil, it may be expected that the results of the testing undertaken would suggest that the DSM column installation process would improve the strength characteristics of the *in situ* soils around the columns. As the soils are mechanically mixed and jet grouted, the energy would likely induce stress onto the *in situ* soils outside the columns, inducing granular realignment of the soils and resulting in lateral compaction. However, whether the results would indicate a positive or negative response was unclear. Previous research had suggested that soil response to DSM in New Zealand cohesive, clayey soil profiles resulted in a positive ground improvement response. This is explored and discussed in more detail in forthcoming sections of this chapter.

6.2 Results Summary Synthesis

6.2.1 Liquefaction Analysis

6.2.1.1 Process Overview

The CPT is used to determine many geotechnical properties of soils. For this research the CPT was used to understand general parameters of cone resistance, sleeve friction, and pore pressure. These outputs enabled the quantification of the liquefaction potential of the *in situ* soils preceding and following installation of DSM columns.

Figures 5-1, 5-2 & 5-3 of Chapter 5 portray the results of the liquefaction analysis at test location Y at the research site (prior to and following ground improvement). The testing was conducted at distances 500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm from the installed column centre and to a target depth of 12 m.

Standing water was assumed to be 3.5 m bgl from the preliminary site investigation (summarised in Chapter 3) which was conducted prior to this research with additional review of the surrounding well data from GNS and EQC. For soils to liquefy they must be saturated, therefore the predicted settlement outputs only assume a liquefiable response from the assumed standing water depth of 3.5 m bgl.

6.2.1.2 Results Summary

The results from the CPT suggest a general decrease in liquefaction potential following DSM column installation from 750 mm outside the column centre. From 750 mm from the column centre, the more distant the test was from the column, the less the decrease in liquefaction potential was observed.

As the DSM column is installed, the stress applied to the *in situ* soils is greatest in the soil closest to the mixing. Therefore, the tests conducted 500 mm from the column centre show an increase in the liquefaction potential when compared to the pre-DSM CPT. This outcome suggests the stress applied to the soil has created a disturbance which results in the destruction of the soil fabric immediately adjacent to the DSM column.

Upon review of relevant literature, it is suggested that over time, as the stress and pore pressure decrease, the soil is likely to become denser and more uniform. This is a limitation on the use of the test results and suggested further research is described in Chapter 7 of this thesis.

The significance of this result suggests that DSM columns do positively affect the *in situ* soils around the columns.

6.2.1.3 Results Synthesis

The first test located 500 mm from the column centre portrayed an increase in predicted settlements over the pre ground improvement test. Since the liquefaction potential is primarily a function of the cone resistance, this suggests the soils directly adjacent to the DSM column have been disturbed and negatively affected causing the destruction of soil fabric. The test within the disturbed soils directly adjacent to the column has predicted the liquefaction potential to increase by approximately 30% when compared to the pre DSM CPT. The destruction of the soil fabric is likely due to stress relaxation of the soils which had been studied by Spira, et al., (2005) in a tunnelling and ground reinforcement context. If testing was conducted around the same column at a similar distance (assuming no soil disturbance would occur) 12 months after construction, previous research from Spira, et al., (2005) suggest that the soil would progressively recover from its disturbed state and become denser. This is an area which could be explored further in future research and will be discussed in more detail in Chapter 7.

In the subsequent testing conducted further from the DSM column centre, a decrease in liquefaction potential was observed. The tests conducted 750 mm away from the column centre portray a decrease of total predicted settlement of approximately 50% compared to the test results prior to DSM installation. As each test becomes more distant from the installed DSM column, the less the predicted decrease in liquefaction potential is observed (Figure 6-1).

The results suggest the lateral confinement within the transition zone caused by DSM installation, influences the *in situ* soils more significantly the closer the soil is to the mixed column. However, within soils effected by the expansion zone (within 50 – 100 mm of the column) the liquefaction potential increases (deteriorate), as suggested above.

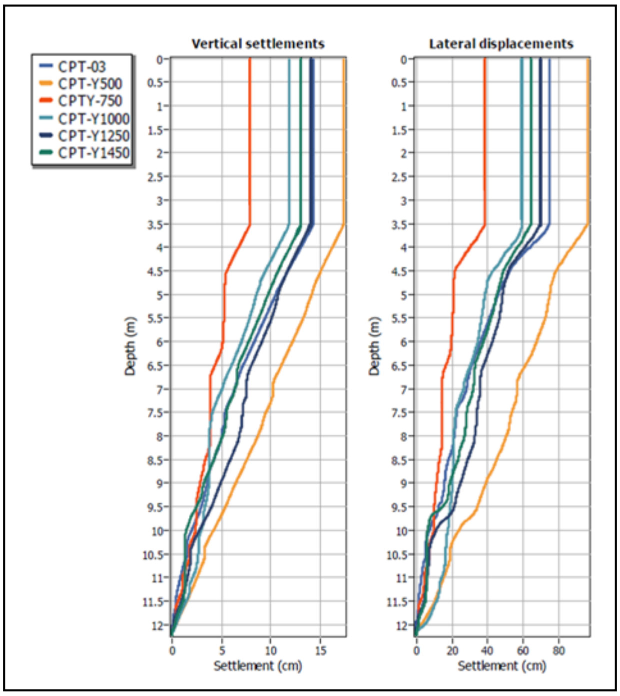


Figure 6-1 – Predicted Liquefaction Potential Plots ULS exported from CLiq at Test Location Y

This CPT testing was conducted in order to understand soil behaviour and to estimate the soil profile's liquefaction response prior to and following DSM installation. Analysis of the *in situ* soil response has suggested that a DSM-soil interaction can be observed and suggests minimised liquefaction potential around DSM columns.

The CPT is a simple way to assess whether the *in situ* soil characteristics have been altered as a function of DSM column installation. By analysing the CPT results, there is a general decrease in liquefaction potential at the research site. Therefore, when DSM columns are installed in a triangular open grid arrangement, positive effects and improvements are likely within the *in situ* soil characteristics in soils approximately 750 mm from the column centre (900 mm diameter column) or 300 mm from the column edge. This confirms the objective of examining whether DSM induces any ground improvement characteristics to *in situ* soils in a Christchurch alluvial context by conducting standardised test methods.

However, these results are taken from three separate locations on a single research site in the northeast quarter of Christchurch City. Further testing and analysis would be required to formulate a strong data set for design to ensure this response is consistent across Christchurch City.

6.2.1.4 Supporting Research

Previous research has used CPT tests to verify the liquefaction mitigation of ground improvement methods within *in situ* soils. Chen & Bailey (2004) indicated that when verification testing was completed within *in situ* soils around stone columns, that an improvement was only observed in more granular soils.

In some cases, with cohesive soil profiles, either no change in soil response or even deterioration was observed. This is similar to what has been observed in this research. From the results of the CPT, liquefaction potential at the research site decreases within sandy soils. However, due to the water table depth being assumed to be below the silty soils, conclusions relating to the CPT response in these soils cannot be made from this research.

6.2.2 Material Index (I_D)

6.2.2.1 Process Overview

The DMT test was implemented to predict a series of *in situ* soil responses following ground improvement. I_D is a soil behaviour classification method which categorises soils from the behavioural response to the DMT blade as it is advanced into the ground.

Marchetti (2001) suggests the Material Index (I_D) provides an indication of soil type. However, this is not necessarily representative of absolute particle size, rather the mechanical behaviour of how the soil is responding to testing. For example, soils may be defined as a sandy silt from sieve analysis or particle distribution, but may be responding to I_D testing as a silty sand depended on the sensitively of the soil and the cut off value adopted for analysis. Thus I_D provides are more accurate representation of how the *in situ* soils will respond to DSM ground improvement.

In Chapter 5, Figure 5-5 shows the results of I_D variations at test location Y (prior to and following ground improvement) at the research site. The testing has been conducted at distances 500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm from the installed column centre.

6.2.2.2 Results Summary

The results of I_D from the DMT test show a general increase in soil stiffness response to DSM column installation. The silty soils within the upper 3.5 m portrayed a less consistent change to DSM compared with the granular soils observed between 3.5 to 8.5 m bgl. However, the further the tests were taken from the DSM columns, the smaller the increase in stiffness that was recorded in the plots.

The clayey soils at 3.2 m bgl portrayed no change in I_D before and after DSM column installation. These layers have likely sheared as the stress from the DSM is applied outside the columns.

Like with the CPT results and other research, the values of I_D have generally improved as a function of DSM. This suggests that DSM columns result in ground improvement of the surrounding *in situ* soils.

6.2.2.3 Results Synthesis

The results portray a noticeable increase in I_D in the test located 500 mm from the DSM column centre. In Figure 6-3, the results from 3.5 m bgl to 7.5 m bgl (green box) have portrayed a general increase in I_D soil stiffness of approximately 35%. As the testing becomes more distant from the installed column, the I_D testing results continue to show a moderate increase in stiffness within the granular soil profile below 3.5 m. At 750 mm from the column centre an approximate increase of 25% was recorded, and at 1250 mm from the column centre, approximately a 10% increase was recorded. Like with the CPT soil response, the further away from the column the tests were taken, the smaller the change in I_D soil stiffness is observed.

These areas of increase are primarily soils of a more granular material. The absolute soil profile derived from the initial site investigation suggests this unit is primarily sand and sandy gravel. *In situ* sandy soils typically have more void space and less (or no) cohesion which enable soils grains to move over each other and increase soil stiffness as a function of ground improvement (Yang, et al., 2010).

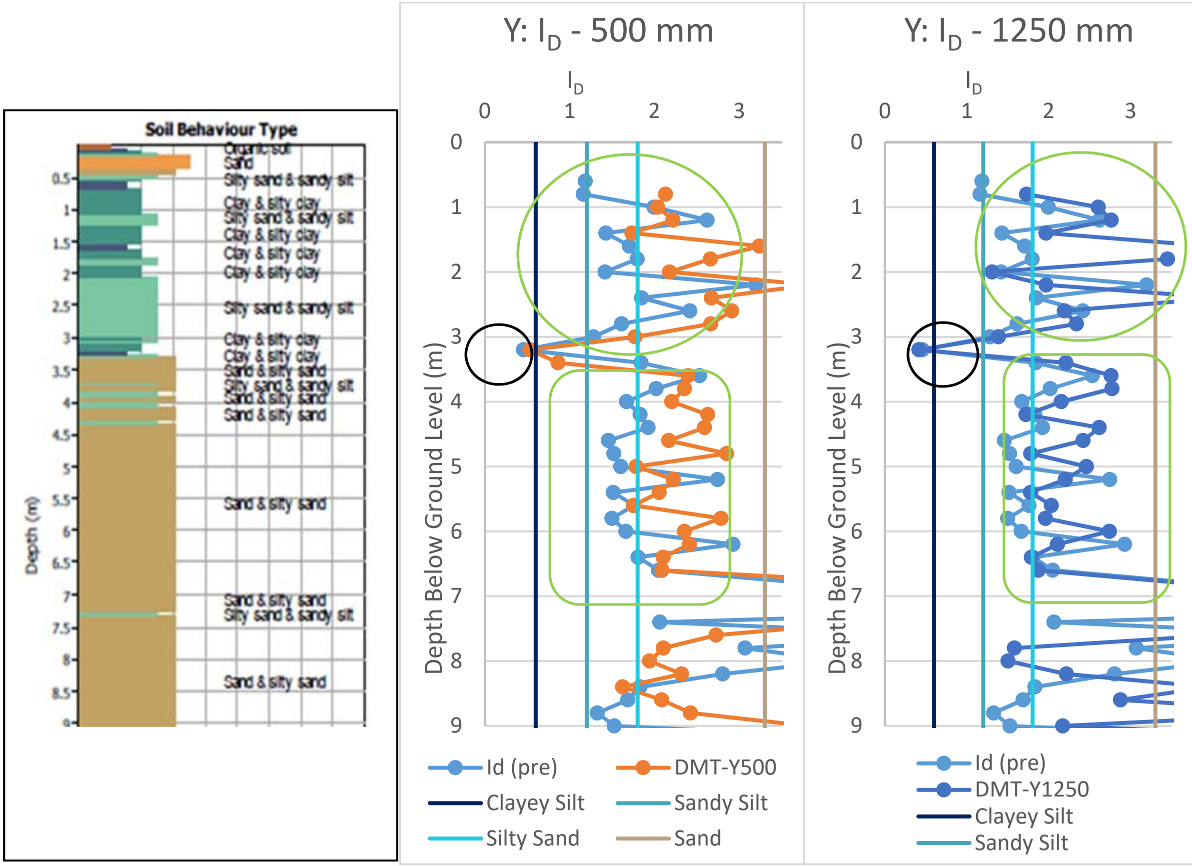


Figure 6-2 – Material index (I_D) versus depth before and after DSM installation.

The layers observed in the deposits (upper 3.5 to 4 m bgl) which have a more cohesive, silty and clayey I_D, did show a particular trend (green circles in Figure 6-3) following DMT testing. Although there is some areas of decreased I_D, this was not consistent across the data set and a consistent soil characteristic change was able to be deduced where an average increase of 20% was observed. More response was recorded in soils within the tests closer to the column (40% increase 500 mm from the column centre), and the further the tests were completed from the column, the less change was recorded (approximate 10% increase 1250 mm from column centre).

The thin deposits logged as clay and silty clay (at between 3 m and 3.5 m bgl) by the CPT soil behaviour type, have induced little to no change in I_D as a result of DSM column installation. Uncompacted cohesive soils tend to have high fines content and minimal void space *in situ* and therefore, as a consequence of jet grouting and mechanical mixing of DSM column installation, *in situ* cohesive soils can have varied changes in I_D. These results are consistent with other research which tests the response cohesive soils have to ground improvement.

The increase in I_D in the sandy and silty soils close to the DSM column (500 mm) is likely a function of increased soil rigidity due to ground improvement. As the DSM columns are installed, the positive volume increase of the DSM process (grout binder injected into and mechanically mixed into the ground) is likely to induce stresses onto the soil and cause the *in situ* soil particles around the DSM column to laterally compact. This results in the increased rigidity of the soil directly adjacent to the column and a progressive decline in I_D the further the soil is from the columns. This confirms the objective of examining whether DSM induces any ground improvement characteristics to *in situ* soils in Christchurch by conducting standardised test methods.

6.2.2.4 Supporting Research

Previous research indicates that I_D likely becomes modified as a function of the installation of different ground improvement methods. However, the densification or rigidity increase is entirely a function of the support and soil type of the surrounding *in situ* soils (Raison, 2004). Therefore, this explains the variation of I_D within the upper clayey deposits compared to the more silty and granular material.

DMT testing completed in Hamilton (New Zealand) within the Hamilton Ash and Puketoka Formations (which are clayey cohesive materials) had positive results for increases in I_D. From these results, Tatarniuk (2014) suggested that in these locations, there is some lateral confinement within *in situ* soils due to DSM column installation. In a comparison to the results derived from the research site, fine grained, more sensitive silty soils and the granular soils did respond relatively consistently. These results are similar to what was observed with the research completed by Chen & Bailey (2004) with stone column ground improvement response in more cohesive soil profiles where varied improvement in I_D was measured and silty and sandy soils portrayed a positive response is soil stiffness.

The DMT is a simple empirical soil behaviour test method selected for the fast, repeatable and comparable outputs which are derived. However, there are disadvantages of the method. A likely weakness is the geotechnical parameters derived are purely based on empirical correlations which were developed in the late 1980's (Long, 2008). Further limitations of this test method are the lack of soil sample recovered, which can often be required for calibration of the results, due to the empirical nature of the method. Subsequent testing may be required to confirm the *in situ* soil profile of the site. Other limitations of the test include they will likely terminate due to practical refusal in dense gravel and boulder rich soils.

6.2.3 Horizontal Stress Index (K_D)

6.2.3.1 Process Overview

The horizontal stress index (K_D) is a soil parameter derived from the DMT test which estimates the *in situ* soils resistance to volumetric change (Monaco & Marchetti, 2007). K_D is the lateral earth pressure a soil can transfer in the horizontal direction and is a function of the soil properties and stress history. As an example, if the same soil is deposited equally, then disturbed and subsequently re-compacted, K_D will vary dramatically between *in situ* and disturbed response.

Figure 6 in Chapter 5 displays the results, pre and post DSM column installation, of the K_D at test location Y. Like with the other DMT tests, the testing has been conducted at distances 500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm from the installed column centre.

6.2.3.2 Results Summary

The results of K_D within the granular soils between 3.5 and 8.5 m bgl portray a general improvement in soil response to DSM column installation. The granular soils have increased stiffness and density as a function of the lateral stress applied to the *in situ* soils from the DSM column installation process. An average of approximately 20% increase was recorded across at the test locations between 3.5 m to 8.5 m bgl.

The silty soils within the upper 3.5 m portrayed minimal change (approximately 10%) following DSM compared with the granular soils observed between 3.5 to 8.5 m bgl (approximately 20%). This is likely due to the cohesive behaviour of the silty materials which generally do not portray stiffness increases through empirical test methods.

The results show a marked decrease of approximately 7% in K_D 500 mm away from the column centre (red box on Figure 6-4) within the granular soils below 3.5 m bgl. The decrease in K_D 500 mm from the DSM column, similarly to the 500 mm CPT result, is likely a function of soil fabric deformation directly adjacent to the DSM column.

Like with the CPT results and other research, the values of K_D have generally increased as a function of DSM. This suggests that DSM column installation results in an improvement of the *in situ* soils around the columns.

6.2.3.3 Results Synthesis

The greatest increase in K_D was recorded within the more granular materials (3.5 m to 8.5 m bgl) 750 mm away from the column centre (green box in Figure 6-4) with an approximate increase of 50% over the original test. This response suggests a positive lateral rigidity increase in soils following DSM. However, like with the CPT and I_D response to DSM, the further away the tests were conducted from the column centre, the less the observed effective increase of K_D. 1000 mm from the column centre an approximate increase of 18% was recorded, and approximately 9% increase was recorded 1450 mm from the column centre. Therefore, the stress applied from the DSM columns is likely to reduce the further the tests are conducted from the DSM column.

However, in the test completed 500 mm from the column centre, the granular soils suggested a similar response to DSM where the results were worse than the initial tests conducted prior to DSM (approximate 7% decrease). This response is likely a function of the soil fabric being disturbed.

Destruction of the soil fabric is commonly attributed due to stress relaxation of the soils (Spira, et al., 2005). Ground improvement using DSM columns typically induces significantly less volumetric strain than other common methods of ground improvement, and therefore less destruction of the soil fabric and re-alignment of soil particles will be observed (Tatarniuk, 2014). However, when relevant literature was investigated, there was no available research which focused and quantified the degree of *in situ* soil fabric relaxation around DSM columns following ground improvement.

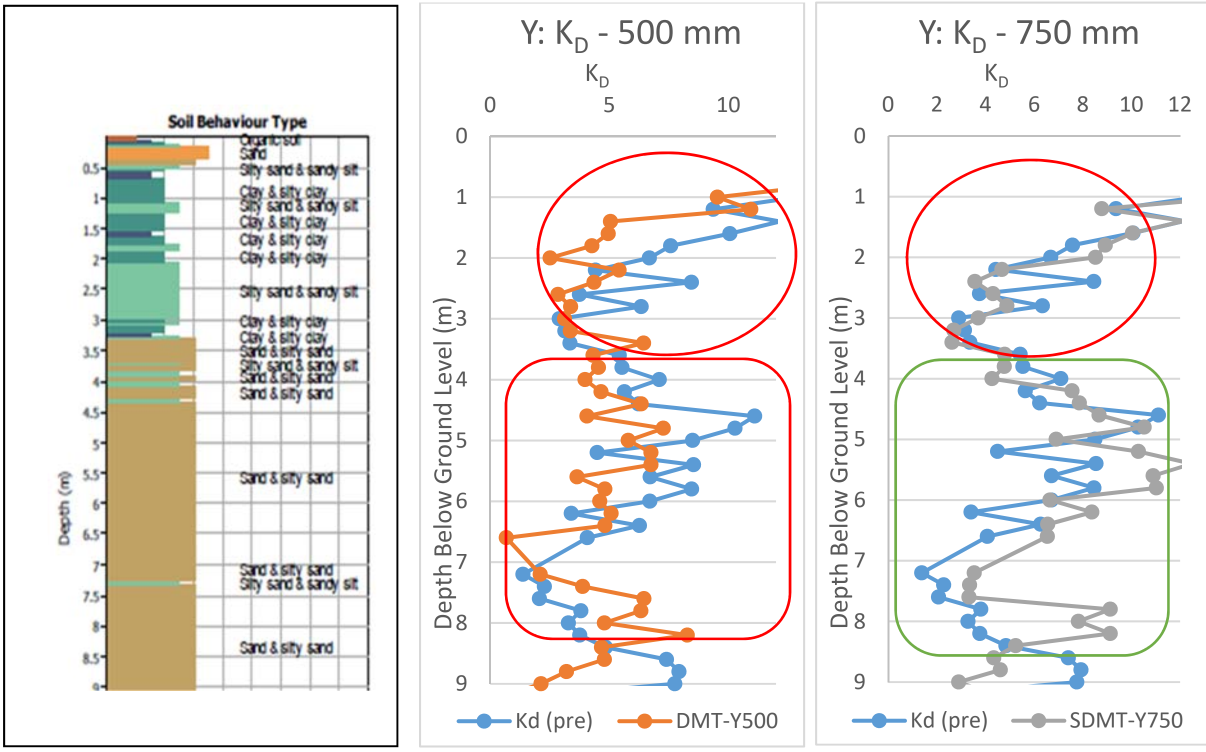


Figure 6-3 – Horizontal Stress Index (K_D) versus depth before and after DSM installation.

The layers observed in the siltier deposits (upper 3.5 to 4 m bgl), which have a more cohesive soil composition, did not show any particular trend in K_D increase following DSM installation (Red circles in Figure 6-4). The cohesive soils typically shear and induce disturbance of the soil fabric as stress from ground improvement is applied. Despite these mixed results of K_D, from reviewing relevant literature, it is believed that the soils are still being improved due to the increased stiffness of the soils recorded by the other empirical test methods.

6.2.3.4 Supporting Research

Previous research by Tatarniuk (2014) had suggested that K_D was used effectively to detect slip faces in overconsolidated clay soils in the Central North Island of New Zealand. It was described that the DMT test (K_D) could quantify the loss of structure in the slip plains. Like with the soil fabric destruction in the soils around DSM columns examined at the research site, the tests conducted directly adjacent to the DSM column had undergone a loss in strength which was measured by K_D in the DMT test. However, the applied stress from DSM is generally lower than other methods of ground improvement (Spira, et al., 2005). Upon research of relevant literature, limited information was found and further research is required to quantify the applied stresses effecting *in situ* soils around DSM columns

As previously discussed, the DMT was selected because of the empirical, fast, repeatable and comparable nature of the test method. A weakness of the test is that the geotechnical parameters derived are purely based on empirical correlations which were developed in the late 1980's (Long, 2008). For calibration of the test, *in situ* soil samples may be required to confirm the soil profile. In addition, the DMT test often will struggle to penetrate dense gravel and boulder rich soils.

6.2.4 Shear Wave Velocity (V_s)

6.2.4.1 Process Overview

The shear wave velocity (V_s) is derived from an adaption to the mechanical DMT test with a seismic module (which incorporates two seismic receivers spaced 0.5 m apart above the DMT Blade) which measures the influence seismic waves have on the *in situ* ground. V_s is estimated by the difference in the distance between each seismic receiver, divided by the arrival delay period from the signal source of impact. V_s is a geotechnical parameter which is strongly influenced by the state of the soil and the composition of the soil fabric.

The results displayed in Figure 5-6, Chapter 5 show the results of the shear wave velocity testing completed at location Y at the research site. Like with the DMT test locations (since the sDMT is an adapted version of the DMT), the testing was conducted at distances of 500 mm, 750 mm, 1000 mm, 1250 mm and 1450 mm from the installed column centre.

6.2.4.2 Results Summary

The results of the V_s plots show that the greatest increase in the granular soils occurs 500 mm from the column centre (approximately 40% across the entire soil profile to 8.5 mbgl) and that the increase in V_S diminishes the further the tests were conducted from the columns. An average of approximately 25% increase in stiffness was recorded 750 mm from the column centre, and approximately 18% increase in V_S was recorded 1450 from the column centre. The silty soil in the upper 3.5 m shows a less consistent response to ground improvement. Within the cohesive soils in the test conducted 500 mm from the column centre, the soils have portrayed some deterioration. Although the results are less consistent, the average increase remains approximately 15% 500 mm from the column centre. This is likely a function of the cohesive soil fabric being disturbed and inducing destruction due to being stressed during the DSM column installation process.

Generally with this method, the faster the V_S, the greater the density and stiffness of the *in situ* soils, and the less prone the soils are to liquefy in a future seismic event.

6.2.4.3 Results Synthesis

It is apparent that there is a greater increase in shear wave velocity (V_s) in soils closer to the column (at 500 mm an approximate increase of 40% was recorded) and progressively, as the tests become more distant from the ground improvement, the V_s increase is less. These tests recorded consistent V_s results within the cohesionless (sandy) soils below 3.5 m bgl before and after DSM. In contrast, where the tests were conducted in the cohesive soils (behaviour types of silts and clays) recorded in the upper 3.5 m, the V_s results were mixed in response which is similar to previous research conducted with stone columns in silty soils.

The lateral confinement of the soil closer to the column is likely a function of the positive volume increase of DSM installation as the cement binder is injected and mechanically mixed into the ground. The additional volume induces additional lateral stress outside the DSM column. Stresses are applied to the *in situ* soil causing a radial displacement and realignment of the soil particles. As this shearing of soil occurs, soil particles move adjacent soil particles out of the way to form an interlocking structure and thus increasing soil density (Spira, et al., 2005). However, as the horizontal stresses diminish the further the tests were from the column, the ability of soil particles to move over each other lessens, and the tapering effect of V_s becomes smaller.

Previous research summarised for deep pile foundations, suggests when a pile is driven into cohesionless soils, densification occurs within the *in situ* soils around the piles at a distance ranging from three to five diameters (Das, 2011). With the V_s results examined in cohesionless soils around DSM columns, densification has increased to at least 1.5 times the diameter of the column (1450 mm) relatively consistently. However, at the research site, the DSM column spacing is 2.9 m centre to centre (approximately 3.2 times the column diameter (900 mm)), so testing completed at a distance greater than 1450 mm from the column centre would likely induce interfered results from the overlapping of the transition zone between the adjacent columns.

When comparing both pre and post DSM column installation, the V_s tends to further increase with depth. This is likely attributed to the increased overburden pressure from the overlying material inducing greater density within the undisturbed soils. Thus, enabling the applied shear stresses during DSM column installation to induce greater confinement at depth in similar cohesionless soils.

Within the more sensitive soils in the upper 3.5 m (Figure 6-5), like with the previous CPT and DMT results, the V_s results are mixed. In some cases, V_s values are less than the pre ground improvement testing results. This was emphasised in the tests performed closer to the installed DSM columns (500 mm and 750 mm) in comparison to the results further from the columns (1000 mm to 1450 mm). Similar to what is described previously, the varied results within the cohesive, near surface soils are likely a function of the destruction of the soil fabric due to DSM. The soil fabric destruction within the cohesive soils is commonly observed in other research of various ground improvement methods.

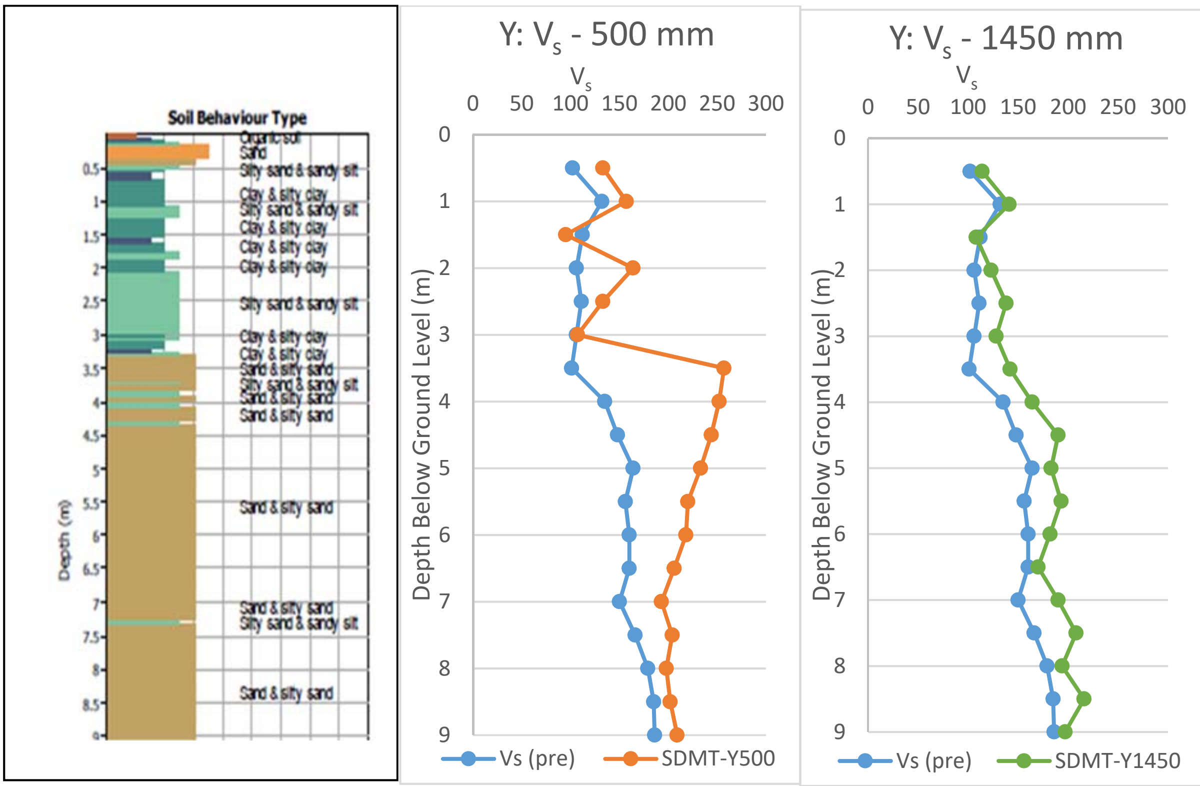


Figure 6-4 – Shear wave velocity (V_s) versus depth before and after DSM installation.

6.2.4.4 Supporting Research

The previous research of Slocombe, Bell, & Baez (2000) testing summarised the response of *in situ* silty and sandy soils to stone column installation. Their results suggest a consistent increase in lateral densification within sands. However, within clayey and silty layers, the densification is less consistent, and in some cases, less than the pre installation testing. The soil response in both cohesive and cohesionless soils is consistent with what was observed at the research site. The tests show that in cohesionless soils there was a consistent increase in V_s. By comparison, test results in cohesive soils portrayed a mixed response in the change of V_s following DSM.

V_s testing was selected for the research as a fast, non-destructive method to measure soil stiffness increases. The sDMT is more sensitive to soil stress history, soil aging, cementation and soil structure changes than many other invasive test methods. However, this method still relies on empirical correlations of data from the broader area and may require soil sampling test methods to provide calibration for this method. Specialised equipment and a skilled operator is required to complete the tests.

6.2.5 Small Strain Shear Modulus (G_0)

6.2.5.1 Process Overview

The small strain modulus (G_0) is directly proportional to the V_s response, the bulk density of the *in situ* soil, which the S-waves are propagating through. G_0 is an important geotechnical parameter used in many different aspects of geotechnical and earthquake engineering, such as liquefaction potential, site specific response and soil structure interaction. G_0 depends on the soil effective stress, void ratio, soil saturation, stress history and the stiffness of the soil skeleton, determined by the interparticle cementation of the soil fabric (Santamarina, et al., 2001).

In this research, to establish a small strain stiffness model, G_0 is derived from V_s in the sDMT test previously described in Chapter 2, Equation 2.14.

The results portrayed in Figure 5-7 of Chapter 5 show the results of G_0, which have been derived from Vs testing, completed at location Y at the research site. Like with the sDMT test locations used for V_s, the testing was conducted at distances of 500 mm, 750 mm, 1000 mm, 1250 mm, and 1450 mm from the installed column centre.

6.2.5.2 Results Summary

Since G_0 is the function of $V_{s,}$ the results depicted in Figure 6-6, have a similar trend to the V_s response to testing summarised in section 6.2.5 above. Therefore, similarly to V_S, the results portray the greatest increase in G_0 within the granular soils below 3.5 m bgl when conducted 500 mm from the column centre. An average increase of approximately 100% was recorded 500 mm from the column centre, and as the testing is conducted further away, the percentage increase diminishes. Approximately 50% increase in stiffness was recorded at 1000 mm from the column centre and approximately 40% increase at 1450 mm from the column centre was recorded.

The silty soil in the upper 3.5 m presents mixed results. The soils which are closer to the column have portrayed a worse result (approximately 40% increase 500 mm from column centre in soils within the upper 3.5 m), and as the tests are conducted further from the columns the soils begin to improve (approximately 50% increase 1000 mm from column centre in soils within the upper 3.5 m). This suggests that the stress induced from the mixing process induces soil fabric destruction within 500 mm from the column centre, but as the tests are conducted further from the column, the soils portray a small improvement.

6.2.5.3 Results Synthesis

The results indicate a marked increase in G_0 across all the test locations conducted at test location Y. It is noted that there is the greatest increase in G_0 in the test 500 mm from the column centre, and as the test locations become more distant from the column, the improvement results reduce as stated previously. This is a function of the stress transferred during DSM installation causing soil particles to shear, realign and become more confined (and thus increase G_0). However, as the shear stress decreases, the results further from the column also decrease and diminish in increase of G_0.

Within the upper 3.5 m (cohesive deposits), like with the V_s results, the results have generally increased in G_0, although some results portray a negative or mixed response to ground improvement. As previously suggested, the soil fabric is likely to have been deformed as a function of the energy during DSM column installation. The soil fabric has likely been disturbed within the cohesive soils, which is commonly seen within various ground improvement methods.

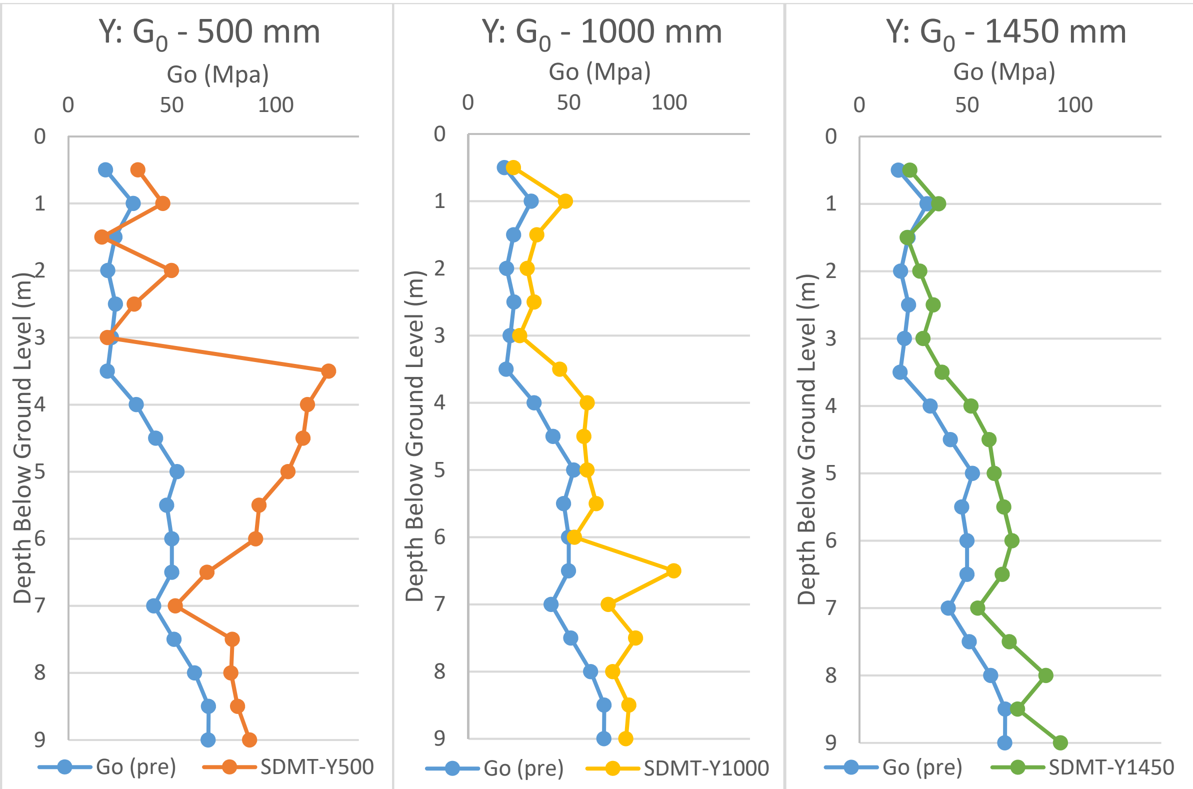


Figure 6-5 – Small Strain Shear Modulus (G_0) versus depth before and after DSM installation.

6.2.5.4 Supporting Research

Previous research conducted to model G_0 from the derivation of V_s has suggested differences and inaccuracies in results. The assumed derivation of G_0 suggests that in saturated soils, using equation 2.14 (Chapter 2), shear waves tend to travel through soils and through water in phase with each other (Monaco & Marchetti, 2007). However, Guadalupe-Torres (2013) believes that shear waves partially follow the fluid motion rather than the soil skeleton, and thus using bulk density (ρ), may result in over estimation and inaccurate results of G_0. They go on to conclude that this may only affect specific soil types in specific areas of the world. For this research, it has been assumed that the bulk density (ρ) of the saturated soil is providing an adequate result.

Like with the V_s testing method, the sDMT is a more sensitive method, which enables fast and accurate characterisation of soil profiles in a non-destructive manner. However, this test still relies on empirical correlations which can skew results without surrounding soil profile data to calibrate the test method. Specialised equipment and a skilled operator are also required to complete the testing correctly.

6.3 Conclusions

Previous research had been analysed and has confirmed the validity of using empirical test methods to measure the soil response to DSM column installation. However, minimal testing had been completed in soils around DSM columns in a Christchurch alluvial context. Chen & Bailey (2004) indicated that verification testing (CPT) was completed in soils around an open grid stone column layout and indicated a positive soil response to ground improvement. Tatarniuk (2014) suggested that when conducting DMT tests in clay soils, the empirical outputs showed a valid response to ground improvement.

The research objectives of this thesis were to assess the soil characteristic changes radially around DSM columns using a series of standardised invasive test methods. The field investigation of this research combined the invasive test methods of CPT, DMT and sDMT, which were used to examine the soil response to DSM in a Christchurch alluvial soil setting. A generally positive response to ground improvement was observed and repeatedly quantified across all the empirical test results at least 750 mm away from the column centre (300 mm away from column edge).

The results from the other test location (location X) portray a similar response to the documented location (location Y). Location Z was unable to be tested following DSM due to time constraints associated with construction. The results of test location X are provided in Appendix L

A positive response is defined as ground improvement within the unmixed soils outside the DSM columns where the soil characteristics have been altered to increase overall soil stiffness and minimise future liquefaction induced settlement. However, some mixed results were observed in the silty materials in the soil profile in the upper 3.5 m bgl. Upon review of relevant research using other ground improvement methods, this outcome was consistent with the results that others have observed.

The results of the invasive tests 500 mm from the column centre showed a varied response. From review of other literature, this is likely a function of the destruction of the soil fabric as stress from the mixing process is applied to soils adjacent to the installed column. Although in some cases the tests deteriorate, research suggests that as the soil ages, the soil will likely begin to restore. The effect of soil ageing on its strength and stiffness would be a beneficial focus for future research.

These results are taken from three separate locations on a single research site in the northeast quarter of Christchurch City. To confirm this research is accurate for a Christchurch alluvial context, further research and testing at multiple sites would be recommended to confirm the response *in situ* soils have to DSM. The invasive test methods are simple and repeatable following the same principles of this research.

In the forthcoming chapter (Chapter 7), the conclusions and further recommendations of this research are explored. The limitations of the research process are summarised and potential future research will be discussed.

7 Conclusions and Further Recommendations

7.1 Research Objectives

The objectives of this research were to examine the confinement radially around DSM columns and to assess whether there is a reduction in the cyclic strain softening and liquefaction potential of the saturated soils following DSM. The benefit of this research is to demonstrate whether DSM provides a secondary benefit of improving the performance of the *in situ* soils in the event of a significant earthquake.

Minimal research or testing had been conducted to assess the response and modification of the soil characteristics in a Christchurch alluvial context around the DSM columns. Therefore, a secondary outcome of this research is that simple, standardised test methods can be used to quantify ground improvement. This lack of investigation provides the justification of this research.

However, with any research, there are limitations and thus potential for future testing and research to establish the performance in a range of alluvial soil types. These are discussed in further detail below.

7.2 Industry Relevance

Having the ability to repeatedly quantify lateral confinement, stiffness and shear strength of the *in situ* soils around DSM columns, would enable contractors and DSM practitioners to have confidence in less invasive and more cost effective test methods. Using these test methods for project quality assurance, would refine the existing processes by minimising the need for trial columns and invasive testing coupled with laboratory analysis, and would provide significant cost savings, reduced construction time and provide a more efficient outcome for future DSM geotechnical projects.

Using standardised, repeatable invasive methods would allow for quantifiable assessment of DSM improving the *in situ* soils around the columns. This, if established as a standard method for DSM verification testing, could be utilised as a regular approach on future DSM projects to confirm the effectiveness of the ground improvement over the existing borehole and laboratory methods.

7.3 Results Summary and Principle Conclusions

The research site was selected to examine the *in situ* soil response to DSM column installation, and to provide conclusions to answer the objectives determined at the beginning of this thesis. This site had been subject to PGA which caused liquefaction induced settlement and lateral spreading during the CES due to saturated near surface alluvial soils.

The field investigation of this research combined invasive test methods of CPT, DMT and sDMT which were completed before and after DSM column installation. The series of tests were selected to establish whether an improvement to the soils around DSM columns did occur, and if so, what soil characteristics of the *in situ* profile were affected the most (Chapter 5). This soil-DSM interaction had previously not been examined in a Christchurch alluvial soil context.

The results of this research show that changes to the soils around DSM columns do exist, and an increase in the *in situ* soil strength and stiffness influences the performance of the soil-DSM system (Chapter 6). This increase in soil rigidity is important for practitioners to understand when quantifying ground improvement using DSM columns, and demonstrates that the method is not limited to just the column mixing.

The liquefaction potential (derived from the CPT) of the soils portrayed a marked decrease in the tests 750 mm to 1450 mm away from the column centre. However, within the soils 500 mm or less from the column centre, an increase in liquefaction potential was recorded. This is likely due to soil fabric destruction during the mixing process.

I_D and K_D (derived from the DMT) portrayed similar increases to the liquefaction response within the granular soils below 3.5 m in tests a minimum of 750 mm away from the columns, where an improvement in ID and KD was observed (Section 5.3 and Section 5.4). However, in the tests completed 500 mm away from the columns, a mixed or even negative response was observed.

Both V_S and G_0 (sDMT) portrayed an increase in soil response due to DSM ground improvement (Section 5.5 and Section 5.6). The cohesive soils (upper 3.5 m) within the test 500 mm from the column centre, again portrayed mixed results. This is likely to the result of soil fabric deformation.

Within all the test types, there was a recorded increase in soil response to DSM in the range from 500 mm to 1450 mm from the column centre. The positive soil responses suggest that in the designed open grid arrangement (0.9 m diameter columns with 2.9 m spacing), that there would likely be a grouping effect. Das (2011) suggests that within a piled foundation, grouping effects can be allowed for in design, with up to a 20% *in situ* soil bending moment and stiffness increase within piles spaced three diameters apart. With DSM, further research would have to be undertaken to examine this grouping effect using laboratory and modelling. However, the results suggest that there is a grouping effect within DSM columns installed in an open grid arrangement in alluvial soils.

7.4 Limitations of Research

Although the research has answered the objectives summarised at the beginning of the thesis, there are some limitations which need to be considered.

The limited data set sourced from one research site in the northeast of Christchurch City is the most significant limitation. Using one research site, with a single alluvial soil profile causes the outcomes of this thesis to become specific to the research site. However, this program was unable to include and test additional sites across Christchurch due to time and cost restraints of the project. The generalisations which are able to be deduced from the single research site provided a sturdy basis for interpretation due to the soil profile being generally representative of Christchurch near surface geology. Further expansion of testing would establish a broad means of comparison between *in situ* soil response around DSM columns. More data would enable a comparison of any outlying responses and allow for a more robust data set.

The results of the tests were in accordance with previous research conducted for DSM column installation and other methods such as stone columns and driven timber piles, thus providing confidence in the outcome. The results of this research offer support to DSM practitioners in regularly using *in situ* empirical test methods to quantify ground improvement in a practical, site to site approach.

Secondly, the testing was only at three separate locations across the single site and was not repeated. Repeating the test procedure in the same locations and by completing more tests across the site would have confirmed the accuracy and repeatability of the test methods and outcomes at the research site. Measurement of the testing at different time intervals could have enabled further research of the soil aging effect that is expected to occur within the disturbed soils directly adjacent to the DSM columns.

Another limitation of the research was that different contractors completed the tests before and after DSM column installation. Although the tests are standardised and validated as effective methods by the American Society for Testing and Materials, the procedures being conducted may have been done inconsistently causing slightly varied results. Human error and variation between operators during completion of the test methods is something that cannot be controlled in the research. However, the outcomes of the tests were comparable and consistent with other research utilising the same testing methods.

7.5 Recommended Future Research

This research is limited by the conventional *in situ* test methods used, rather than more controlled methods which could be used in a laboratory setting.

The outputs derived from this research have shown that several alternative testing methods could be used to establish the property changes of the soils. This research provides a starting point towards understanding how DSM influences soils in an alluvial context.

As suggested above, further *in situ* testing would provide for a more comprehensive data set and help to confirm the results of this research. Additional research, both in Christchurch and around the world, would enable more consistent quantification of soil response to DSM.

This research would be enhanced by modelling using finite methods to better understand the predicted response of soils to DSM columns acting as a group. Modelling how the soils respond would highlight the grouping effect of the DSM columns and could provide a general rule of thumb like Das (2011) has suggested for piling.

Laboratory testing could be conducted to further analyse the soil response to DSM at a more detailed level to define how the DSM process affects alluvial soils. Analysis could be conducted to understand the disturbed soil zone adjacent to the expansion zone and the transition zone around the columns in different alluvial soils typical to Christchurch. This would allow for an understanding of the expected disturbance with soils around DSM columns and quantify the group effect of the DSM columns.

Finally, it would be useful to have research done that focuses on the soil aging affect or soil stress relaxation following the completion of DSM columns. Typically, the stress and strain of soils change over time around piles, and has been examined successfully in cohesionless and cohesive soils. However, there is no relevant research which summarises this phenomena in literature relevant to a DSM in a Christchurch alluvial soil context. It would be useful to understand whether the disturbed soil fabric, which was observed in the tests 500 mm from the column centre, improves and becomes restored over time in a DSM context.

Completing this additional research could help geotechnical DSM practitioners to use standardised invasive test methods to measure and confirm ground improvement in future DSM projects.

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Appendix A – CPT, DMT and sDMT Testing Standards

Cone Penetrometer Test (CPT) Standards:

- ASTM standard D5778-12

Flat Dilatometer Test (DMT) Standards:

- ASTM D6635 – 15
- Eurocode 7, Part 3 (1997)
- ISSMGE TC16 (2001)

Seismic Flat Dilatometer Test (sDMT) Standards:

- ASTM D6635 – 15
- Eurocode 7, Part 3 (1997)
- ISSMGE TC16 (2001)

Appendix B –DMT and sDMT Reduction Formulae

DMT Reduction Formulae (Marchetti S. M., Monaco, Totani, & Calabrese, 2001):

SYMBOL	DESCRIPTION	BASIC DMT REDUCTION FORMULAE	
p_0	Corrected First Reading	$p_0 = 1.05\,(A - Z_M + \Delta A) - 0.05\,(B - Z_M - \Delta B)$	Z_M = Gage reading when vented to atm. If ΔA & ΔB are measured with the same gage used for current readings A & B, set $Z_M = 0$ (Z_M is compensated)
p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$	
I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$	u_0 = pre-insertion pore pressure
K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{v0}$	σ'_{v0} = pre-insertion overburden stress
E_D	Dilatometer Modulus	$E_D = 34.7\,(p_1 - p_0)$	E_D is NOT a Young's modulus E. E_D should be used only AFTER combining it with K_D (Stress History). First obtain $M_{DMT} = R_M\,E_D$, then e.g. $E \approx 0.8\,M_{DMT}$
K_0	Coeff. Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$	for $I_D < 1.2$
OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5\,K_D)^{1.56}$	for $I_D < 1.2$
c_u	Undrained Shear Strength	$c_{u,DMT} = 0.22\,\sigma'_{v0}\,(0.5\,K_D)^{1.25}$	for $I_D < 1.2$
Φ	Friction Angle	$\Phi_{safe,DMT} = 28° + 14.6°\log K_D - 2.1°\log^2 K_D$	for $I_D > 1.8$
c_h	Coefficient of Consolidation	$c_{h,DMTA} \approx 7\ \mathrm{cm}^2 / t_{flex}$	t_{flex} from A-log t DMT-A decay curve
k_h	Coefficient of Permeability	$k_h = c_h\,\gamma_w / M_h \quad (M_h \approx K_0\,M_{DMT})$	
γ	Unit Weight and Description	(see chart in Fig. 16)	
M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M\,E_D$ if $I_D \le 0.6$ $\quad R_M = 0.14 + 2.36 \log K_D$ if $I_D \ge 3$ $\quad R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $\quad R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ with $R_{M,0} = 0.14 + 0.15\,(I_D - 0.6)$ if $K_D > 10$ $\quad R_M = 0.32 + 2.18 \log K_D$ if $R_M < 0.85$ $\quad$ set $R_M = 0.85$	
u_0	Equilibrium Pore Pressure	$u_0 = p_2 = C - Z_M + \Delta A$	In free-draining soils

Appendix C – Geotechnical Investigation: Hand Augers

FILE NAME:	Willowlea Retirement Home	**DATE:**	23/07/2013
FILE NUMBER:	130850	**WEATHER:**	Fine
SITE LOCATION:	27 Shirley Road, Shirley	**OPERATOR:**	MDS

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Test Bore Results

Depth(m)	Bore Hole at Test 3	WT
0.10		
0.20	Topsoil	
0.30		
0.40		
0.50		
0.60		
0.70		
0.80		
0.90		
1.00		
1.10	Light Brown Silt	
1.20		
1.30		
1.40		
1.50		
1.60		
1.70		
1.80		
1.90		
2.00	Brown & Orange	
2.10	Sandy Silt	
2.20		
2.30		
2.40		
2.50		
2.60	Wet Orange & Brown	
2.70	Clay	
2.80		
2.90		
3.00		

Depth(m)	Bore Hole at Test 10	WT
0.10		
0.20		
0.30	Topsoil	
0.40		
0.50		
0.60		
0.70		
0.80	Grey & Brown Silt	
0.90		
1.00		
1.10		
1.20		
1.30	Brown Sandy Clay	
1.40		
1.50		
1.60		
1.70		
1.80	Damp Bown & Orange Sandy Silt	
1.90		
2.00		
2.10	Damp Brown Clayish	
2.20	Sand	
2.30		
2.40	Wet Brown Clayish Sand	
2.50		
2.60		
2.70		
2.80	Wet Blue Clayish Sand	
2.90		
3.00		

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FILE NAME: Willowlea Retirement Home **DATE:** 23/07/2013

FILE NUMBER: 130850 **WEATHER:** Fine

SITE LOCATION: 27 Shirley Road, Shirley **OPERATOR:** MDS

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Test Bore Results

Depth(m)	Bore Hole at Test 11	WT
0.10	Topsoil	
0.20		
0.30		
0.40	Brown Silt	
0.50		
0.60		
0.70		
0.80	Brown & Orange Sandy Silt	
0.90		
1.00		
1.10	Damp Brown Clay	
1.20		
1.30		
1.40		
1.50		
1.60		
1.70		
1.80		
1.90		
2.00		▼
2.10	Wet Brown Sandy Clay	
2.20		
2.30		
2.40		
2.50		
2.60		
2.70		
2.80		
2.90		
3.00		

Depth(m)	Bore Hole at Test X	WT
0.10		
0.20		
0.30		
0.40		
0.50		
0.60		
0.70		
0.80		
0.90		
1.00		
1.10		
1.20		
1.30		
1.40		
1.50		
1.60		
1.70		
1.80		
1.90		
2.00		
2.10		
2.20		
2.30		
2.40		
2.50		
2.60		
2.70		
2.80		
2.90		
3.00		

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FILE NAME:	Willowlea Retirement Home	**DATE:**	23/07/2013
FILE NUMBER:	130850	**WEATHER:**	Fine
SITE LOCATION:	27 Shirley Road, Shirley	**OPERATOR:**	MDS

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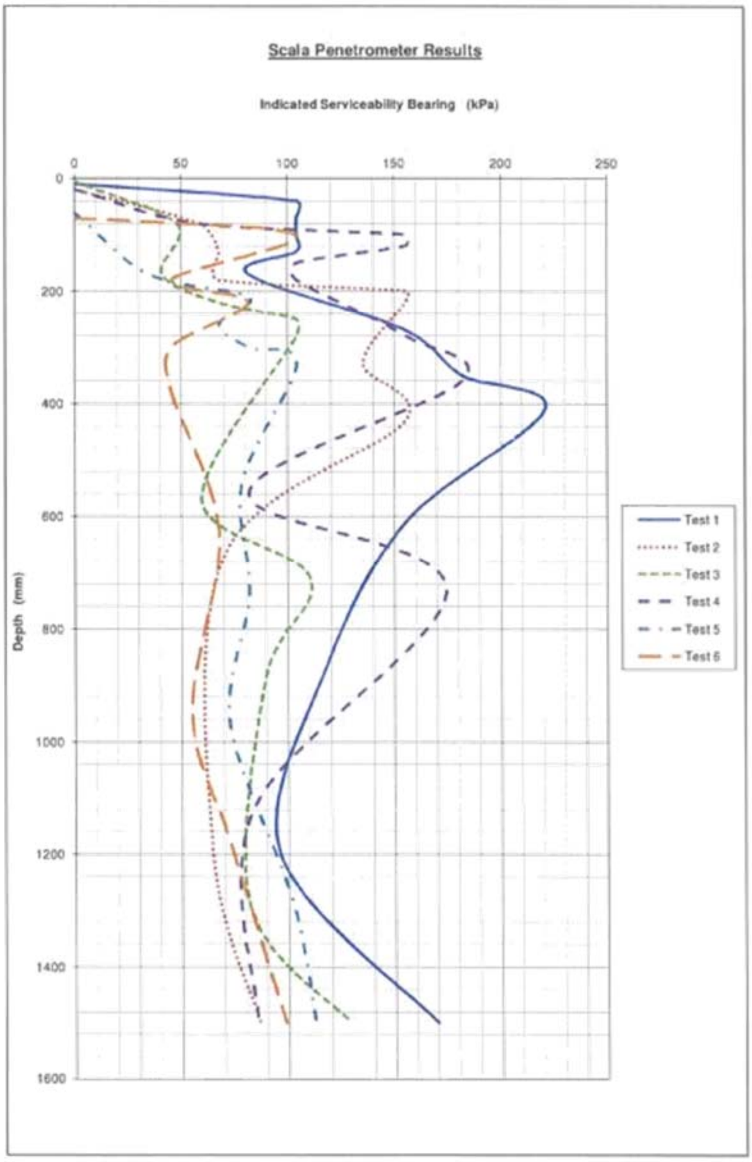
Penetrometer Graph - Bearing over Depth

Technical Category Three (Blue)



FILE NAME: Willowlea Retirement Home **DATE:** 23/07/2013

FILE NUMBER: 130850 **WEATHER:** Fine

SITE LOCATION: 27 Shirley Road, Shirley **OPERATOR:** MDS

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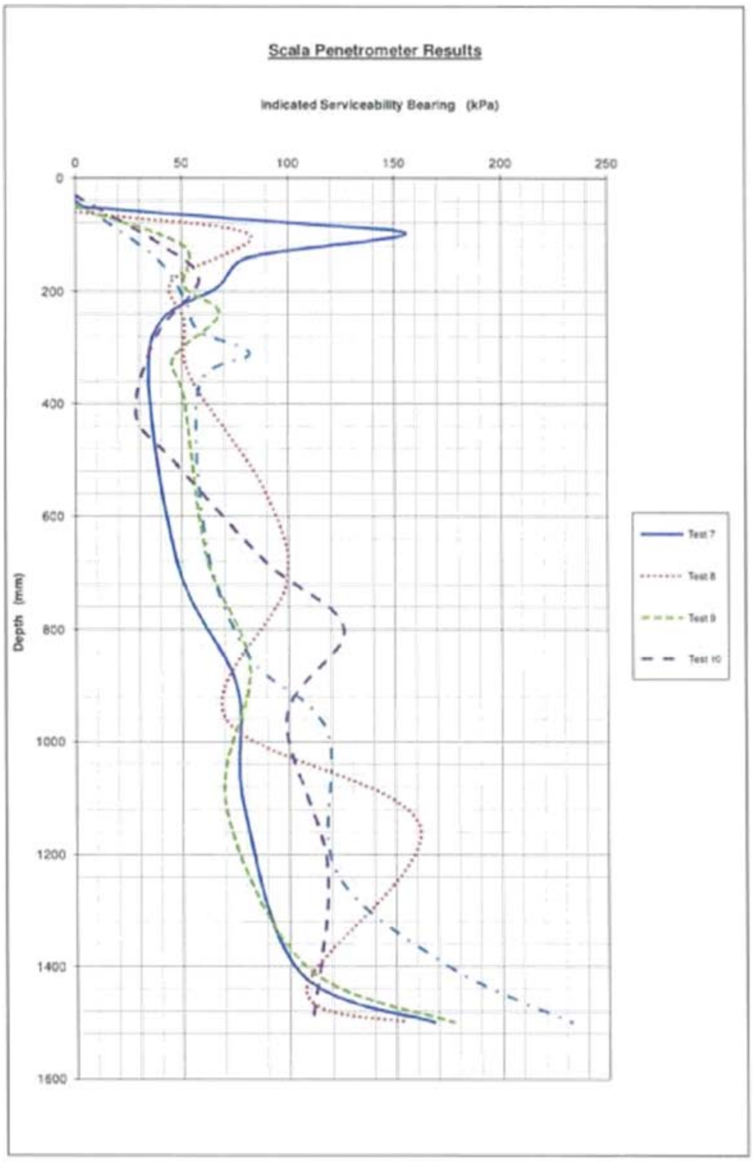
Penetrometer Graph - Bearing over Depth



FILE NAME: Willowlea Retirement Home | **DATE:** 23/07/2013

FILE NUMBER: 130850 | **WEATHER:** Fine

SITE LOCATION: 27 Shirley Road, Shirley | **OPERATOR:** MDS

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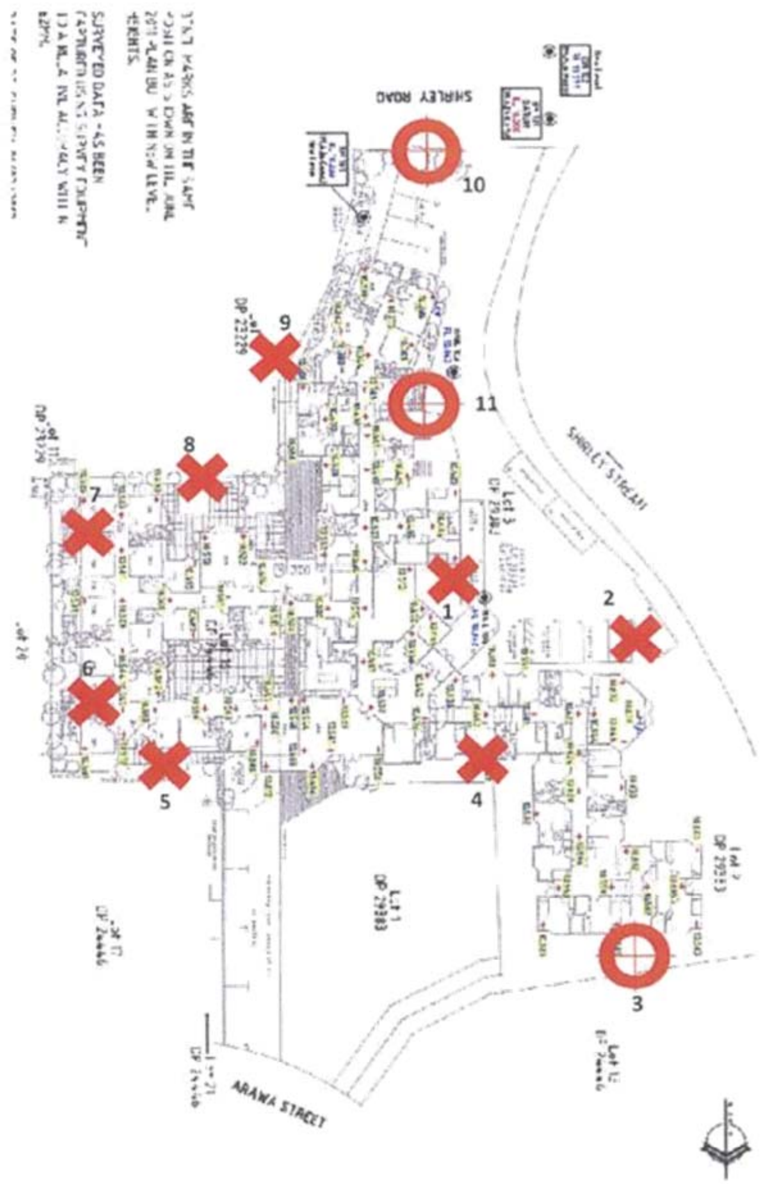
Site Plan

Indicates a site where only a penetrometer test was carried out

Indicates a site where both a penetrometer test and a test bore were carried out



Appendix D – Geotechnical Investigation: Test Pits

coffey

Engineering Log - Excavation

Excavation ID.	TP_01
sheet:	1 of 1
project no.	GENZAUCK16420AA
date excavated:	11 Dec 2014
date completed:	11 Dec 2014
logged by:	Raquel Miller
checked by:	H. MacMurray

client: *Hiway Geotechnical*

principal: -

project: *Willowlea*

location: *27 Shirley Road, Shirley, Christchurch*

position: Not Specified | surface elevation: Not Specified | pit orientation:

equipment type: >6t Excavator Track | excavation method: | excavation dimensions: 2.4 m long 1.5 m wide

excavation information — method, support, penetration (1 2 3), water, samples & field tests, RL (m), depth (m)

material substance

depth (m)	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	vane shear (kPa)	structure and additional observations
0.0–0.3	ML	**SILT**: low plasticity, dark brown, some rootlets and decomposed material present. Slight organic odour.	M			TOPSOIL
0.3–1.2	ML	**SILT**: low plasticity, dark brown, some rootlets and decomposed material present. Trace fine angular gravel and fine grained sand. Bricks and brick fragments present.				FILL
1.2–2.3	ML	**Sandy SILT**: low plasticity, brown, with orange mottling and staining. Some rootlets and decomposed material present.	W			SPRINGSTON FORMATION
2.3–3.55	SM	**Silty SAND**: fine grained, poorly graded, brown, with orance mottling. Trace rootlets and decomposed organic material present. 3.0 m: becoming blue-grey.	S			
3.55–4.2	SP	**SAND**: fine to medium grained, blue-grey.				
4.2		Test pit TP_01 terminated at 4.2 m Collapse				

Water: 11/12/14 (approx. 3.2 m)

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0

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method
- N natural exposure
- X existing excavation
- BH backhoe bucket
- B bulldozer blade
- R ripper
- E excavator

support
- N none
- S shoring

penetration
1 2 3 — no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- U## undisturbed sample ##mm diameter
- D disturbed sample
- B bulk disturbed sample
- E environmental sample
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- VS vane shearpeak/remouded (uncorrected kPa)
- R refusal

classification symbol & soil description
based on Unified Classification System

moisture
- D dry
- M moist
- W wet
- W_p plastic limit
- W_L liquid limit

consistency / relative density
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

Engineering Log - Excavation

Excavation ID.	*TP_02*
sheet:	1 of 1
project no.	*GENZAUCK16420AA*
date excavated:	*11 Dec 2014*
date completed:	*11 Dec 2014*
logged by:	*Raquel Miller*
checked by:	*H. MacMurray*

client: *Hiway Geotechnical*

principal: -

project: *Willowlea*

location: *27 Shirley Road, Shirley, Christchurch*

position: Not Specified | surface elevation: Not Specified | pit orientation:

equipment type: >6t Excavator Track | excavation method: | excavation dimensions: 2.3 m long 1.5 m wide

depth (m)	water	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	structure and additional observations
		ML	SILT: non plastic, dark brown, some rootlets and sub-angular to angular gravel present. Minor fine sand.	M	TOPSOIL
		ML			FILL
		ML	SILT: non plastic, dark brown, some rootlets and sub-angular to angular gravel present. Minor sand and brick fragments.		
1.0			Sandy SILT: non plastic to low plasticity, pale brown, with some orange mottling present. Sand is fine grained. Trace rootletes and decomposed organic material.	W	SPRINGSTON FORMATION
2.0		SP	Silty SAND: fine grained, pale brown, trace rootlets present. becoming blue-grey.		
3.0	11/12/14	SP	SAND: fine to medium grained, blue-grey, minor silt present.	S	
4.0			Test pit TP_02 terminated at 3.7 m Collapse		
5.0					
6.0					
7.0					

method
- N natural exposure
- X existing excavation
- BH backhoe bucket
- B bulldozer blade
- R ripper
- E excavator

support
- N none
- S shoring

penetration
- no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- U## undisturbed sample ##mm diameter
- D disturbed sample
- B bulk disturbed sample
- E environmental sample
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- VS vane shearpeak/remouded (uncorrected kPa)
- R refusal

classification symbol & soil description
based on Unified Classification System

moisture
- D dry
- M moist
- W wet
- W_p plastic limit
- W_L liquid limit

consistency / relative density
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

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Engineering Log - Excavation

Excavation ID.	***TP_03***
sheet:	1 of 1
project no.	***GENZAUCK16420AA***

client:	***Hiway Geotechnical***	date excavated:	***11 Dec 2014***
principal:	-	date completed:	***11 Dec 2014***
project:	***Willowlea***	logged by:	***Raquel Miller***
location:	***27 Shirley Road, Shirley, Christchurch***	checked by:	***H. MacMurray***

position: Not Specified | surface elevation: Not Specified | pit orientation:

equipment type: >6t Excavator Track | excavation method: | excavation dimensions: 1.8 m long 1.5 m wide

method	support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	vane shear (kPa)	structure and additional observations
								ML	**SILT**: non plastic, dark brown, some rootlets present. Minor fine grained sand and fine sub-angular to angular gravel.	M			TOPSOIL
								ML	**Sandy SILT**: non plastic, dark brown, sand is fine grained. Some rootlets and fine sub-angular to angular gravel. Trace brick fragments.				FILL
						1.0		SP	**Silty SAND**: fine grained, pale brown, with orange mottling. Trace rootlets present.	W			SPRINGSTON FORMATION
						2.0			becoming blue-grey.				
			11/12/14			3.0		SP	**SAND**: fine to medium grained, blue-grey, trace silt.	S			
						4.0			Test pit TP_03 terminated at 3.8 m Collapse				
						5.0							
						6.0							
						7.0							

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method
- N natural exposure
- X existing excavation
- BH backhoe bucket
- B bulldozer blade
- R ripper
- E excavator

support
- N none
- S shoring

penetration
no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- U## undisturbed sample ##mm diameter
- D disturbed sample
- B bulk disturbed sample
- E environmental sample
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- VS vane shear peak/remouded (uncorrected kPa)
- R refusal

classification symbol & soil description
based on Unified Classification System

moisture
- D dry
- M moist
- W wet
- W_p plastic limit
- W_L liquid limit

consistency / relative density
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

Appendix E – Geotechnical Investigation: Machine Bore Holes

coffey

Engineering Log - Borehole

Borehole ID.	BH_01
sheet:	1 of 3
project no.	GENZAUCK16420AA
date started:	10 Dec 2014
date completed:	10 Dec 2014
logged by:	Raquel Miller
checked by:	H. MacMurray

client: *Hiway Geotechnical*

principal: -

project: *Willowlea*

location: *27 Shirley Road, Shirley, Christchurch*

position: Not Specified — surface elevation: 0.00 m (Datum Not Specified) — angle from horizontal: 90°

drill model: , Track mounted — casing diameter : 200 mm — vane id.:

drilling information / **material substance**

method & support	samples & field tests	water	RL (m)	depth (m)	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	structure and additional observations
SD / N			0			Asphalt	M		ASPHALT
					GP	Sandy GRAVEL: fine to medium grained, poorly graded, sub-angular to angular, dark brown, sand is medium to coarse grained. Some non plastic silt present.			FILL
					GM	Silty GRAVEL: fine to medium grained, poorly graded, angular, non plastic, dark brown, trace rootlets.	W	F	SPRINGSTON FORMATION
					ML	Sandy SILT: non plastic, dark brown, sand is fine grained, trace rootlets. 0.65 m: becoming low plastic silt.			
			-1	1.0	ML	SILT: low plasticity, pale brown, with some orange mottling. Trace fine sand.			
	SPT 2, 2, 1, 2, 1, 2 N=6				ML	Sandy SILT: non plastic, pale brown, with some orange mottling. Trace rootlets.			
			-2	2.0	SM	Silty SAND: fine to medium grained, poorly graded, non plastic, pale brown.	S	L	
	SPT 2, 1, 3, 2, 3, 1 N=9	10/12/14	-3	3.0					
			-4	4.0	SP	SAND: fine to medium grained, blue-grey, some silt.			
	SPT 1, 1, 0, 2, 2, 2 N=6		-5	5.0		5.0 m: becoming medium grained. 5.3 m: becoming coarse grained.			
	SPT 1, 1, 1, 2, 3, 4 N=10		-6	6.0					
			-7	7.0	GP	Sandy GRAVEL: fine grained, sub-rounded to sub-angular, blue-grey, sand is coarse grained.			
					SP	SAND: coarse grained, blue-grey.			
	SPT 0, 1, 0, 2, 2, 2 N=6				GP	Sandy GRAVEL: fine grained, sub-rounded to sub-angular, blue-grey, sand is coarse grained.			

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method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling
* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud — N nil
C casing

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

coffey

Engineering Log - Borehole

Borehole ID.	BH_01
sheet:	2 of 3
project no.	GENZAUCK16420AA
date started:	10 Dec 2014
date completed:	10 Dec 2014
logged by:	Raquel Miller
checked by:	H. MacMurray

client: *Hiway Geotechnical*

principal: -

project: *Willowlea*

location: *27 Shirley Road, Shirley, Christchurch*

position: Not Specified | surface elevation: 0.00 m (Datum Not Specified) | angle from horizontal: 90°

drill model: , Track mounted | casing diameter : 200 mm | vane id.:

method & support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	vane shear (kPa)	structure and additional observations
SD / N							SP	**SAND**: fine to medium grained, blue-grey, trace rootlets and decomposed organic material.	S	L		SPRINGSTON FORMATION
			SPT 1, 0, 1, 2, 2, 2 N=7	-9	9.0							
				-10	10.0		GP	**Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, blue-grey, sand is coarse grained.		MD		
			SPT 1, 0, 1, 3, 5, 5 N=14									
				-11	11.0		SW	**SAND**: fine to coarse grained, blue-grey.				
			SPT 1, 1, 2, 4, 7, 9 N=22	-12	12.0							
				-13	13.0							
			SPT 1, 3, 3, 5, 7, 10 N=25	-14	14.0							
			SPT 1, 2, 1, 2, 3, 6 N=12	-15	15.0			15.45 m: becoming poorly graded, fine to medium.				

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling
* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
C casing
N nil

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

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coffey

Engineering Log - Borehole

Borehole ID.	**BH_01**
sheet:	3 of 3
project no.	**GENZAUCK16420AA**
date started:	**10 Dec 2014**
date completed:	**10 Dec 2014**
logged by:	**Raquel Miller**
checked by:	**H. MacMurray**

client: ***Hiway Geotechnical***

principal: -

project: ***Willowlea***

location: ***27 Shirley Road, Shirley, Christchurch***

position: Not Specified | surface elevation: 0.00 m (Datum Not Specified) | angle from horizontal: 90°

drill model: , Track mounted | casing diameter : 200 mm | vane id.:

method & support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	vane shear (kPa)	structure and additional observations
SD / N				-16			SW	**SAND**: fine to coarse grained, blue-grey. *(continued)*	S	MD		**SPRINGSTON FORMATION**
			SPT 3, 3, 5, 7, 8, 12 N=32							D		
				-17	17.0							
			SPT 2, 4, 6, 8, 9, 9 N=32	-18	18.0							
				-19	19.0							
			SPT 7, 6, 7, 7, 9, 11 N=34									
				-20	20.0			Borehole BH_01 terminated at 19.95 m Target depth				
				-21	21.0							
				-22	22.0							
				-23	23.0							

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method
- AD auger drilling*
- AS auger screwing*
- HA hand auger
- W washbore
- SD sonic drilling
- * bit shown by suffix
- e.g. AD/T
- B blank bit
- T TC bit
- V V bit

support
- M mud
- N nil
- C casing

penetration: no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- B bulk disturbed sample
- D disturbed sample
- E environmental sample
- SS split spoon sample
- U## undisturbed sample ##mm diameter
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- VS vane shear; peak/remouded (kPa)
- R refusal
- HB hammer bouncing

classification symbol & soil description: based on Unified Classification System

moisture
- D dry
- M moist
- W wet
- S saturated
- Wp plastic limit
- Wl liquid limit

consistency / relative density
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

coffey

Engineering Log - Borehole

Borehole ID.	**BH_02**
sheet:	1 of 3
project no.	**GENZAUCK16420AA**
date started:	**10 Dec 2014**
date completed:	**10 Dec 2014**
logged by:	**Raquel Miller**
checked by:	**H. MacMurray**

client: ***Hiway Geotechnical***

principal: -

project: ***Willowlea***

location: ***27 Shirley Road, Shirley, Christchurch***

position: Not Specified | surface elevation: 0.00 m (Datum Not Specified) | angle from horizontal: 90°

drill model: , Track mounted | casing diameter : 200 mm | vane id.:

drilling information | **material substance**

Method & support: SD, N. Water: 10/12/14 (water level ~2.5 m).

Samples & field tests:

depth (m)	samples & field tests
~1.5	SPT 0, 0, 0, 1, 1, 1 N=3
~3.0	SPT 0, 0, 0, 0, 0, 0 N=0
~4.5	SPT 3, 1, 2, 3, 2, 2 N=9
~6.0	SPT 4, 7, 5, 7, 8, 5 N=25
~7.5	SPT 2, 1, 3, 3, 3, 2 N=11

classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	structure and additional observations
ML	**SILT**: low plasticity, dark brown-black, some rootlets and decomposed material present. Trace fine angular gravel, with some fine grained sand. Slight organic odour.	M		TOPSOIL
ML	**SILT**: low plasticity, dark brown-black, some rootlets and decomposed material. Trace fine angular gravel and fine grained sand. Trace brick fragments.			FILL
ML	**Sandy SILT**: low plasticity, pale brown, with some orange mottling present. Trace rootlets and decomposed organic material.		S	SPRINGSTON FORMATION
ML-MH	**SILT**: medium plasticity, brown-orange, with orange staining. Trace clay.			
ML	**Sandy SILT**: low plasticity, brown, with orange mottling and staining. Sand is fine grained.	W		
SM	**Silty SAND**: fine grained, brown, with orange mottling and staining.	S		
ML	**Sandy SILT**: low plasticity, brown, with orange mottling. Trace rootlets. Sand is fine grained. 2.85 m: large root present.			
Pt	**PEAT**: dark brown, fibrous with larger rootlets present.			
SP	**SAND**: fine to medium grained, blue-grey, trace rootlets present.			
OL	**SILT**: low plasticity, blue-grey and dark brown, rootlets and decomposed organic material present. Slight organic odour.			
ML-MH	**SILT**: low to medium plasticity, blue-grey, some rootlets and decomposed organic material present. Slight organic odour.			
OL	**SILT**: low plasticity, blue-grey, rootlets and decomposed organic material present. Slight organic odour.			
GM	**Silty GRAVEL**: fine to medium grained, sub-rounded to sub-angular, non plastic, brown and blue-grey, some wood fragments and rootlets present. 5.1 m: large piece of wood present. Approx 100mm in length.		MD	
SP	**Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, blue-grey, coarse sand. Some non plastic silt and rootlets present.			
SP	**SAND**: coarse grained, blue-grey, some wood fragments present and fine to medium sub-rounded to sub-angular gravel.			
GP	**Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, blue-grey, sand is coarse grained.			

RL (m): 0, -1, -2, -3, -4, -5, -6, -7 | depth (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0

vane shear (kPa): 50, 100, 150, 200

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling
* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
C casing
N nil

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

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Engineering Log - Borehole

Borehole ID.	**BH_02**
sheet:	2 of 3
project no.	**GENZAUCK16420AA**
date started:	**10 Dec 2014**
date completed:	**10 Dec 2014**
logged by:	**Raquel Miller**
checked by:	**H. MacMurray**

client: **Hiway Geotechnical**

principal: -

project: **Willowlea**

location: **27 Shirley Road, Shirley, Christchurch**

position: Not Specified — surface elevation: 0.00 m (Datum Not Specified) — angle from horizontal: 90°

drill model: , Track mounted — casing diameter : 200 mm — vane id.:

method & support	samples & field tests	RL (m)	depth (m)	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	structure and additional observations
SD, N				GP	**Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, blue-grey, sand is coarse grained. *(continued)* 8.4 m: wood fragments present.	S	MD	SPRINGSTON FORMATION
	SPT 1, 3, 4, 5, 7, 12 N=28	-9	9.0	SW	**SAND**: fine to coarse grained, blue-grey. 9.45 m: becoming coarse grained. Trace fine to medium sub-rounded gravel.			
		-10	10.0		10.0 m: becoming fine to medium grained.			
	SPT 4, 3, 3, 3, 3, 7 N=16	-11	11.0	SP	**Gravelly SAND**: coarse grained, sub-rounded to sub-angular, blue-grey, fine grained gravel.			
				SP	**SAND**: fine to medium grained, blue-grey.			
	SPT 1, 1, 1, 3, 3, 4 N=11	-12	12.0		11.9 m: becoming coarse grained. 12.45 m: becoming fine to medium grained.			
		-13	13.0		12.9 m: becoming medium to coarse grained. 13.2 m: becoming fine to medium.			
	SPT 2, 3, 5, 6, 7, 7 N=25	-14	14.0					
	SPT 1, 0, 1, 2, 3, 4 N=10	-15	15.0					

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method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling
* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud — N nil
C casing

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

coffey

Engineering Log - Borehole

Borehole ID.	**BH_02**
sheet:	3 of 3
project no.	**GENZAUCK16420AA**

client:	*Hiway Geotechnical*	date started:	**10 Dec 2014**
principal:	-	date completed:	**10 Dec 2014**
project:	*Willowlea*	logged by:	**Raquel Miller**
location:	*27 Shirley Road, Shirley, Christchurch*	checked by:	**H. MacMurray**

position: Not Specified | surface elevation: 0.00 m (Datum Not Specified) | angle from horizontal: 90°
drill model: , Track mounted | casing diameter : 200 mm | vane id.:

drilling information: method & support	penetration	water	samples & field tests	RL (m)	depth (m)	material substance: graphic log	classification symbol	material description — SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components	moisture condition	consistency / relative density	vane shear (kPa)	structure and additional observations
SD / N				-16			SP	SAND: fine to medium grained, blue-grey. *(continued)*	S	MD		SPRINGSTON FORMATION
			SPT 2, 3, 5, 5, 9, 13 N=32	-17	17.0							
			SPT 2, 5, 5, 7, 5, 9 N=26	-18	18.0							
				-19	19.0							
			SPT 4, 3, 4, 4, 5, 7 N=20									
				-20	20.0			Borehole BH_02 terminated at 19.95 m Target depth				
				-21	21.0							
				-22	22.0							
				-23	23.0							

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method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling
* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
C casing
N nil

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
WI liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

coffey

Engineering Log - Borehole

client:	*Hiway Geotechnical*	Borehole ID.	***BH_03***
principal:	-	sheet:	1 of 3
project:	*Willowlea*	project no.	***GENZAUCK16420AA***
location:	*27 Shirley Road, Shirley, Christchurch*	date started:	***11 Dec 2014***
		date completed:	***11 Dec 2014***
		logged by:	***Raquel Miller***
		checked by:	***H. MacMurray***

position: Not Specified | surface elevation: 0.00 m (Datum Not Specified) | angle from horizontal: 90°

drill model: , Track mounted | casing diameter : 200 mm | vane id.:

drilling information — method & support: SD, N; water: 11/12/14 (at approx. 3.0 m)

material substance

RL (m)	depth (m)	samples & field tests	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	structure and additional observations
0	0.0		ML	**Sandy SILT**: non plastic, dark brown, minor orange mottling. Sand is fine grained. some rootlets and fine subrounded to subangular gravel.	D	L	CONCRETE
			ML		M		FILL
			SM	**Sandy SILT**: non plastic, pale brown, orange mottling present. Sand is fine grained. Trace rootlets.			SPRINGSTON FORMATION
-1	1.0	SPT 0, 1, 1, 1, 2, 1 N=5		**Silty SAND**: fine grained, poorly graded, pale brown, orange mottling present. Non plastic silt. Trace rootlets present. 1.25 m: slight organic odour. 1.3 m: becoming blue-grey. Thin lenses of decomposed organic material present.	W		
-2	2.0		SP	**SAND**: fine to medium grained, brown, some silt.	S		
-3	3.0	SPT 0, 0, 1, 1, 2, 1 N=5	SM	**Silty SAND**: fine grained, poorly graded, brown, orange mottling present. Non plastic silt. 3.45 m: becoming blue-grey.			
			SP	**SAND**: medium grained, blue-grey, minor silt present. Thin lenses of organic material present. 3.9 m: becoming fine to medium grained.			
-4	4.0			4.1 m: becoming medium grained.		MD	
		SPT 2, 2, 3, 3, 4, 5 N=15		4.5 m: thin gravelly sand layer, approx 0.05m thick. Gravel is fine, subrounded. Sand becoming coarse grained.			
-5	5.0			5.0 m: becoming coarse grained. Some thin low plastic silt lenses present.			
				5.6 m: becoming medium grained.			
-6	6.0	SPT 2, 1, 3, 3, 3, 6 N=15					
-7	7.0		SP	**Gravelly SAND**: coarse grained, blue-grey, gravel is fine to medium, sub-rounded to sub-angular.			
			SP	**SAND**: medium to coarse grained, blue-grey.			
		SPT 0, 1, 2, 2, 3, 5 N=12					

vane shear (kPa): 50, 100, 150, 200 — remoulded, peak

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling

* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
C casing
N nil

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

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Engineering Log - Borehole

Borehole ID.	**BH_03**
sheet:	2 of 3
project no.	**GENZAUCK16420AA**
date started:	**11 Dec 2014**
date completed:	**11 Dec 2014**
logged by:	**Raquel Miller**
checked by:	**H. MacMurray**

client: **Hiway Geotechnical**

principal: -

project: **Willowlea**

location: **27 Shirley Road, Shirley, Christchurch**

position: Not Specified — surface elevation: 0.00 m (Datum Not Specified) — angle from horizontal: 90°

drill model: , Track mounted — casing diameter : 200 mm — vane id.:

drilling information / **material substance**

method & support	samples & field tests	RL (m)	depth (m)	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	structure and additional observations
SD N				SP	**SAND**: medium to coarse grained, blue-grey. *(continued)*		MD	**SPRINGSTON FORMATION**
				SP	**Gravelly SAND**: coarse grained, blue-grey, gravel is fine to medium, sub-rounded to sub-angular.			
	SPT 2, 3, 4, 3, 4, 5 N=16	-9	9.0	SP	**SAND**: medium to coarse grained, blue-grey, some fine sub-rounded to sub-angular gravel present.			
	SPT 4, 6, 9, 13, 14, 14 N=50	-10	10.0	SP	**Gravelly SAND**: coarse grained, blue-grey, gravel is fine to medium, sub-rounded to sub-angular.		VD	
	SPT 1, 2, 2, 5, 5, 7 N=19; SPT 1, 2, 3, 5, 8, 9 N=25; SPT 2, 2, 4, 4, 6, 11 N=25	-11 to -15	11.0 to 15.0	SP	**SAND**: medium to coarse grained, blue-grey, some fine sub-rounded to sub-angular gravel present. 11.6 m: becoming fine to medium grained. Minor silt present. 12.45 m: becoming medium grained. No silt present.		MD	

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method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling
* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud — N nil
C casing

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

coffey

Engineering Log - Borehole

Borehole ID.	BH_03
sheet:	3 of 3
project no.	GENZAUCK16420AA
date started:	11 Dec 2014
date completed:	11 Dec 2014
logged by:	Raquel Miller
checked by:	H. MacMurray

client: *Hiway Geotechnical*

principal: -

project: *Willowlea*

location: *27 Shirley Road, Shirley, Christchurch*

position: Not Specified | surface elevation: 0.00 m (Datum Not Specified) | angle from horizontal: 90°

drill model: , Track mounted | casing diameter : 200 mm | vane id.:

method & support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	vane shear (kPa)	structure and additional observations
SD / N				-16			SP	**SAND**: medium to coarse grained, blue-grey, some fine sub-rounded to sub-angular gravel present. *(continued)*		MD		**SPRINGSTON FORMATION**
			SPT 4, 5, 7, 8, 11, 11 N=37							D		
				-17	17.0							
								17.6 m: becoming fine to medium grained.				
			SPT 2, 3, 5, 7, 8, 13 N=33	-18	18.0							
								18.45 m: becoming medium grained.				
				-19	19.0							
			SPT 15, 10, 9, 9, 12, 10 N=40									
				-20	20.0			Borehole BH_03 terminated at 19.95 m Target depth				
				-21	21.0							
				-22	22.0							
				-23	23.0							

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method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore
SD sonic drilling

* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
C casing
N nil

penetration
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

classification symbol & soil description
based on Unified Classification System

moisture
D dry
M moist
W wet
S saturated
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

Appendix F – Geotechnical Investigation: CPTs

N

KEY

CPT 1 Cone Penetrometer Test

Note: Not to scale; boundaries and locations are approximate only

aurecon
Aurecon New Zealand Limited
Unit 1
150 Cavendish Road
PO Box 1061
Christchurch - New Zealand
Telephone: +64 3 366 0821
Facsimile: +64 3 379 6955
Email: christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

Client: Willowlea Senior CareLtd	Figure 3 Geotechnical Test Locations		Paper Size: A4
Project: Willowlea Senior Care			Revision: 1
By: LFS	Date: October 2011	Job Number: 219038-002	

CPT ANALYSIS NOTES

Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance (q_C) and friction ratio (f_R) only. No normalisation for overburden stress is applied. Cone tip resistance measured with the piezocone is corrected with measured pore pressure (u_C).

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988).

- high susceptibility
- medium susceptibility
- low susceptibility

High susceptibility is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium susceptibility is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Low susceptibility is all other cases.

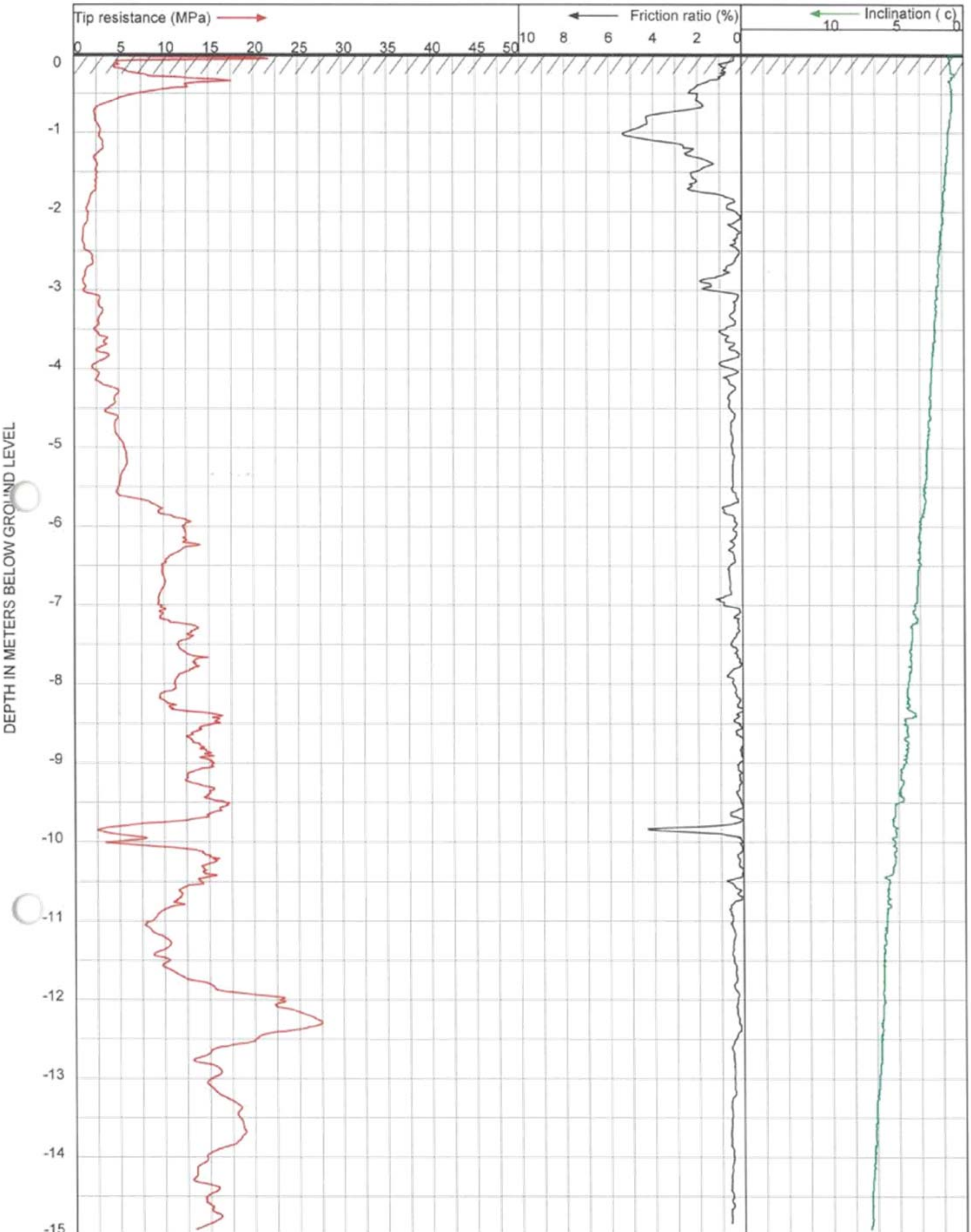
Relative Density (D_R)

Based on the method of Baldi et. al. (1986) from data on normally consolidated sand.

Undrained Shear Strength (S_U)

Derived from the bearing capacity equation using $S_U = (q_C - \sigma_{VO})/15$.

MCMILLAN
DRILLING SERVICES

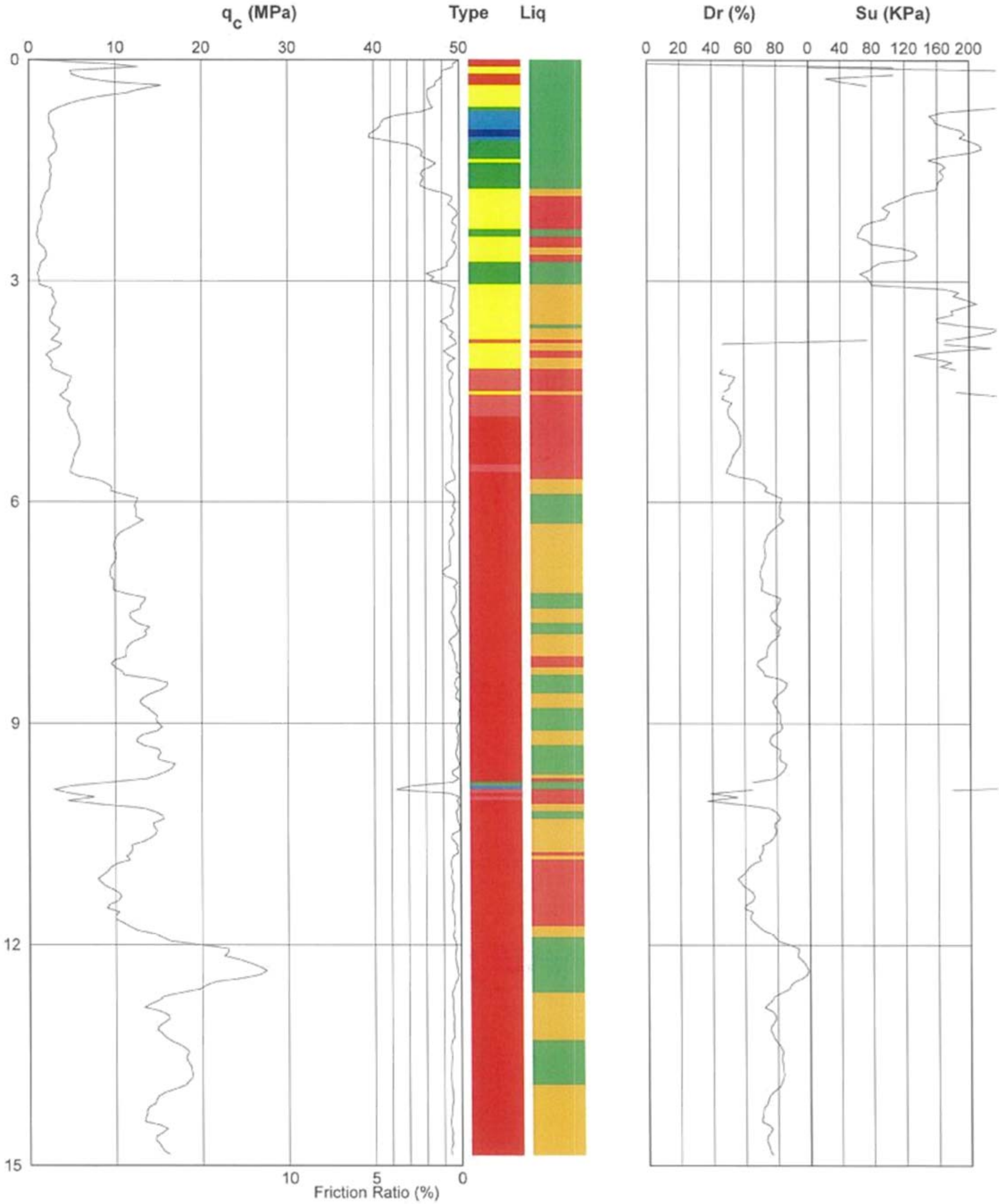


CLIENT : Aurecon Ltd
LOCATION : 27 Shirley Road, Christchurch
DATE : 17-9-2011
OPERATOR : D Keown
REMARK 1 : CPT001
REMARK 2 : Target depth

JOB # : 9115
TEST #: CPT001

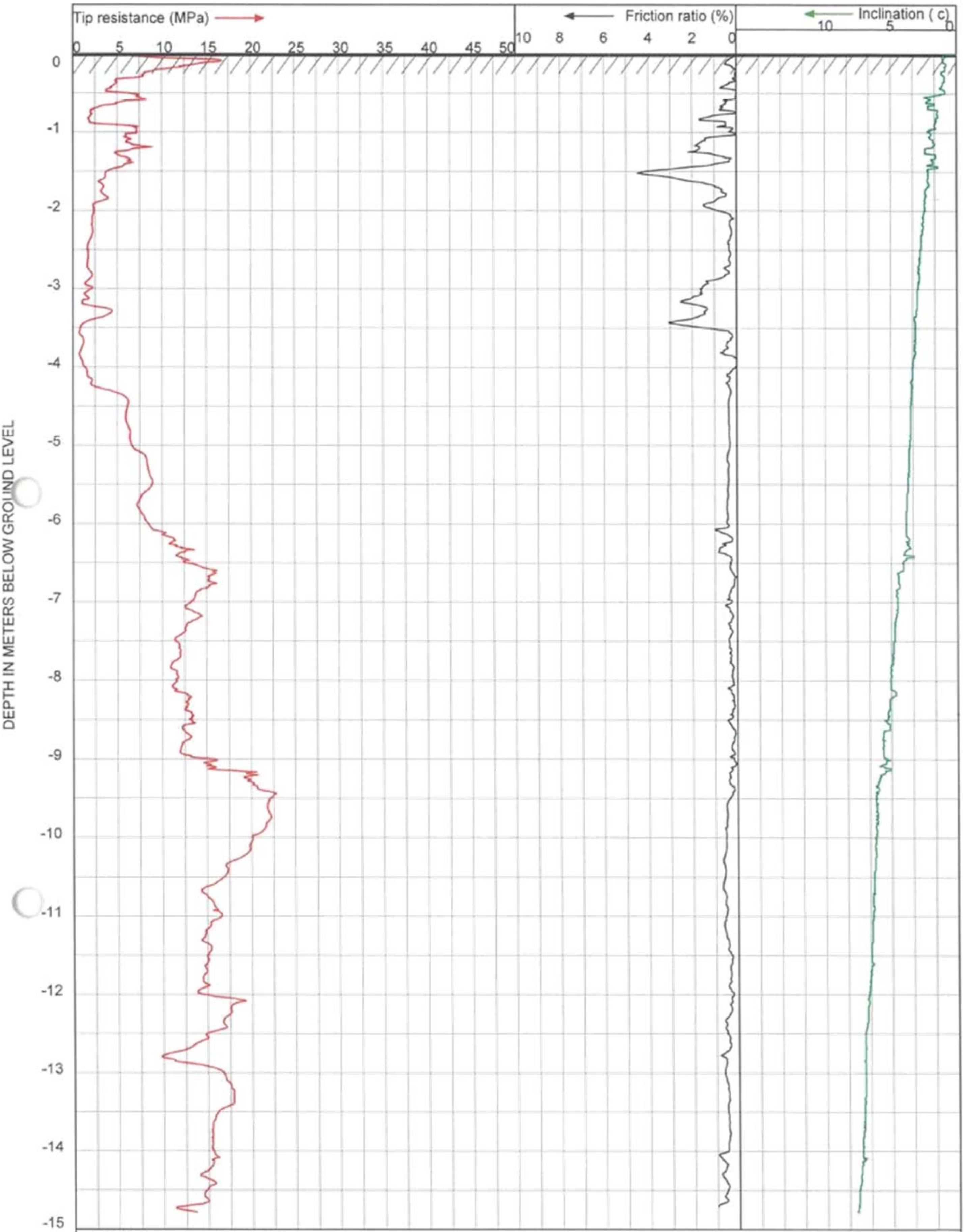
McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	9115	Date:	17-9-2011
CPT No:	CPT001	Operator:	D Keown
Project:	Aurecon Ltd	Remark:	Target depth
Location:	27 Shirley Road, Christchurch		

McMILLAN
DRILLING SERVICES



CLIENT : Aurecon Ltd
LOCATION : 27 Shirley Road, Christchurch
DATE : 17-9-2011
OPERATOR : D Keown
REMARK 1 : CPT002
REMARK 2 : Effective Refusal

JOB # : 9115
TEST # : CPT002

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120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
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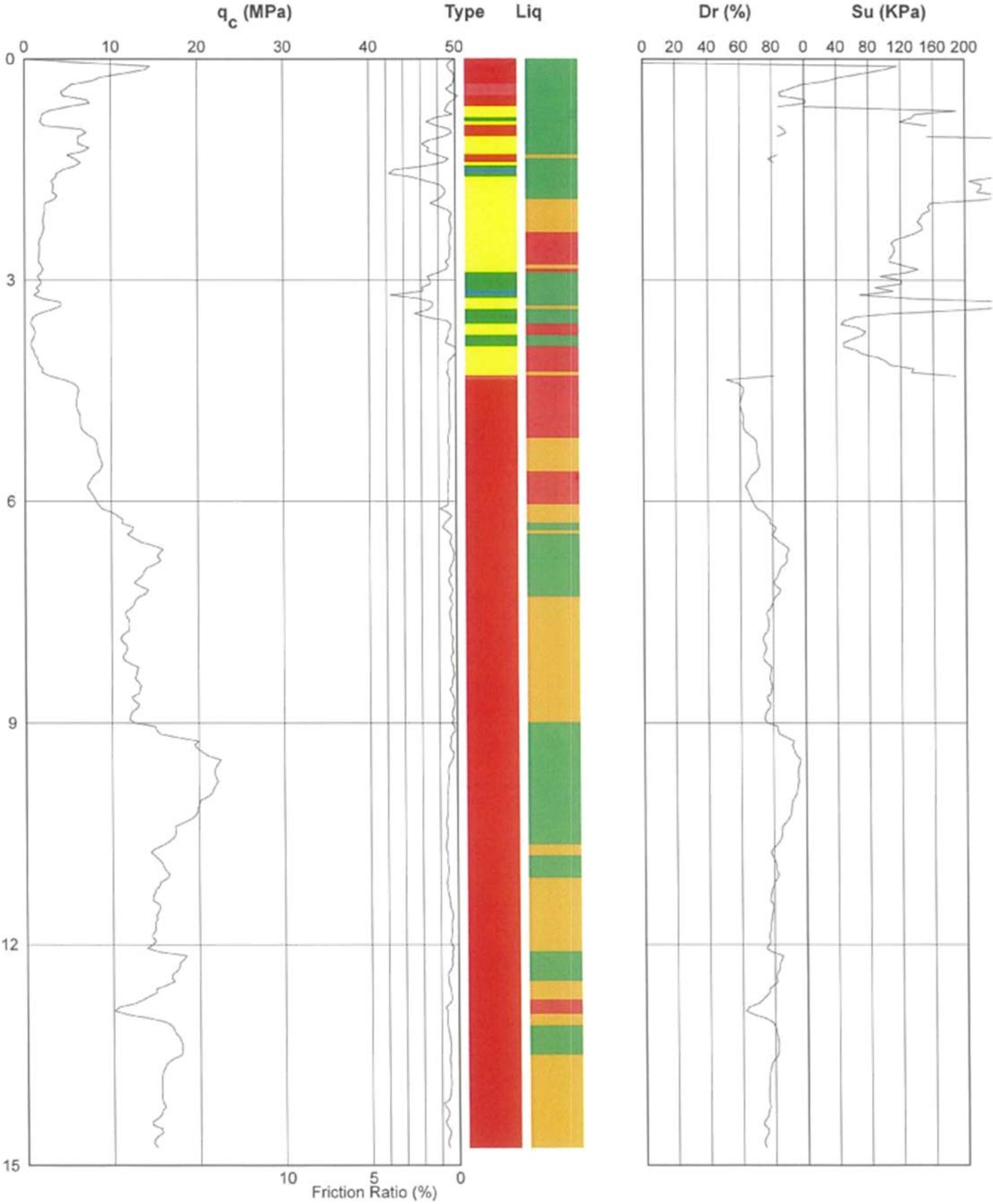
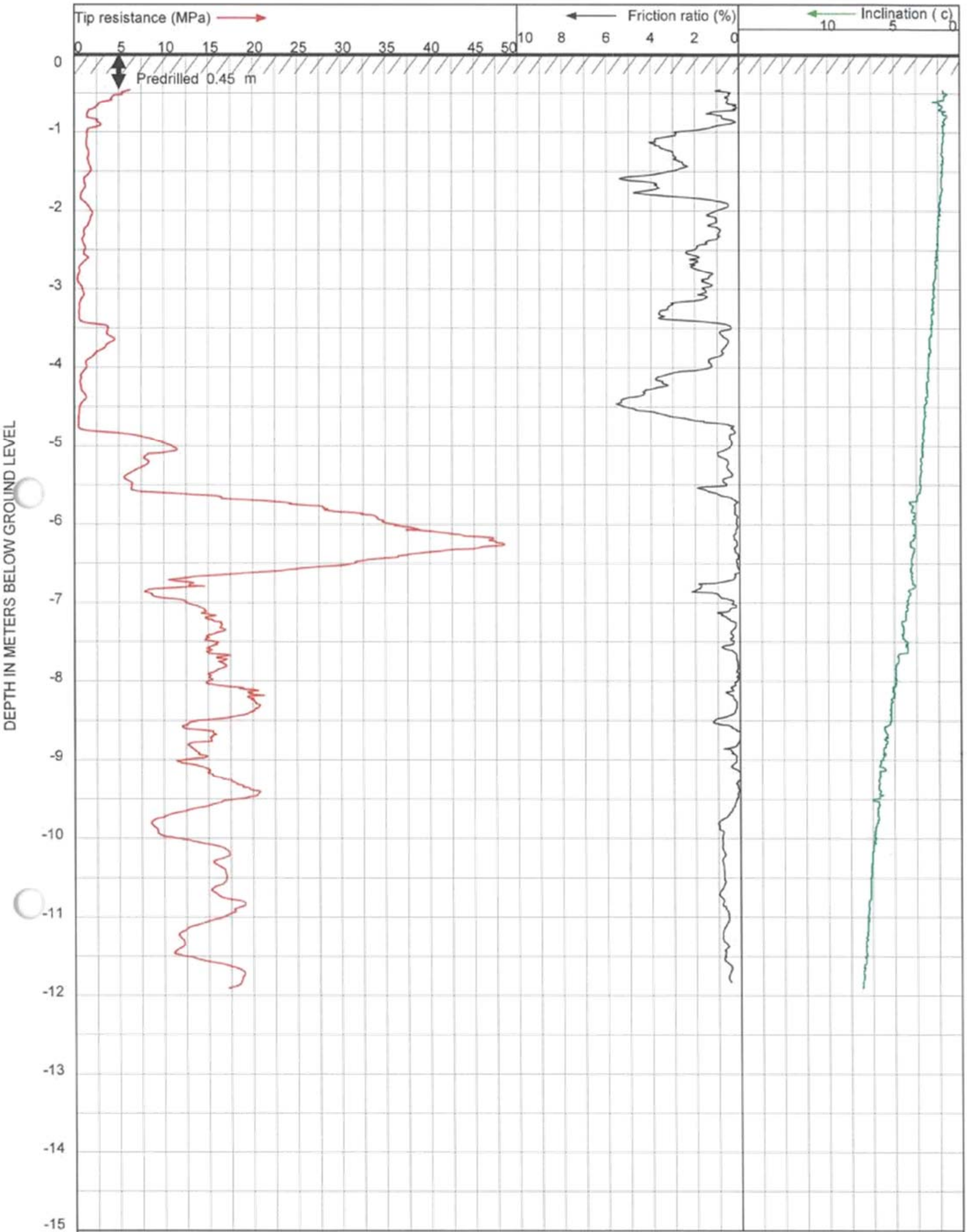
PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No: 9115
CPT No: CPT002
Project: Aurecon Ltd
Location: 27 Shirley Road, Christchurch

Date: 17-9-2011
Operator: D Keown
Remark: Effective Refusal

McMILLAN DRILLING SERVICES

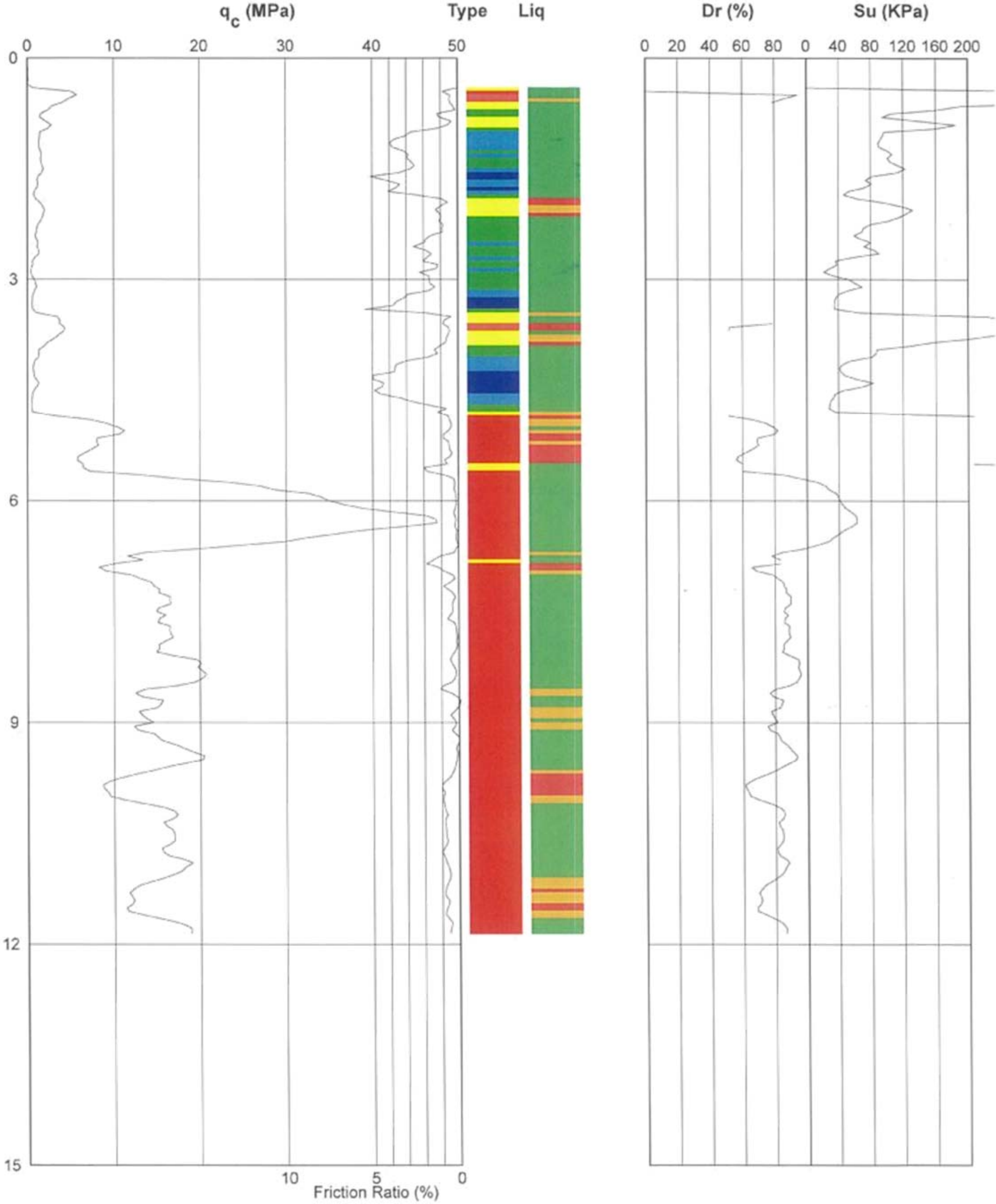


CLIENT : Aurecon Ltd
LOCATION : 27 Shirley Road, Christchurch
DATE : 17-9-2011
OPERATOR : D Keown
REMARK 1 : CPT003
REMARK 2 : Effective Refusal

JOB # : 9115
TEST # : CPT003

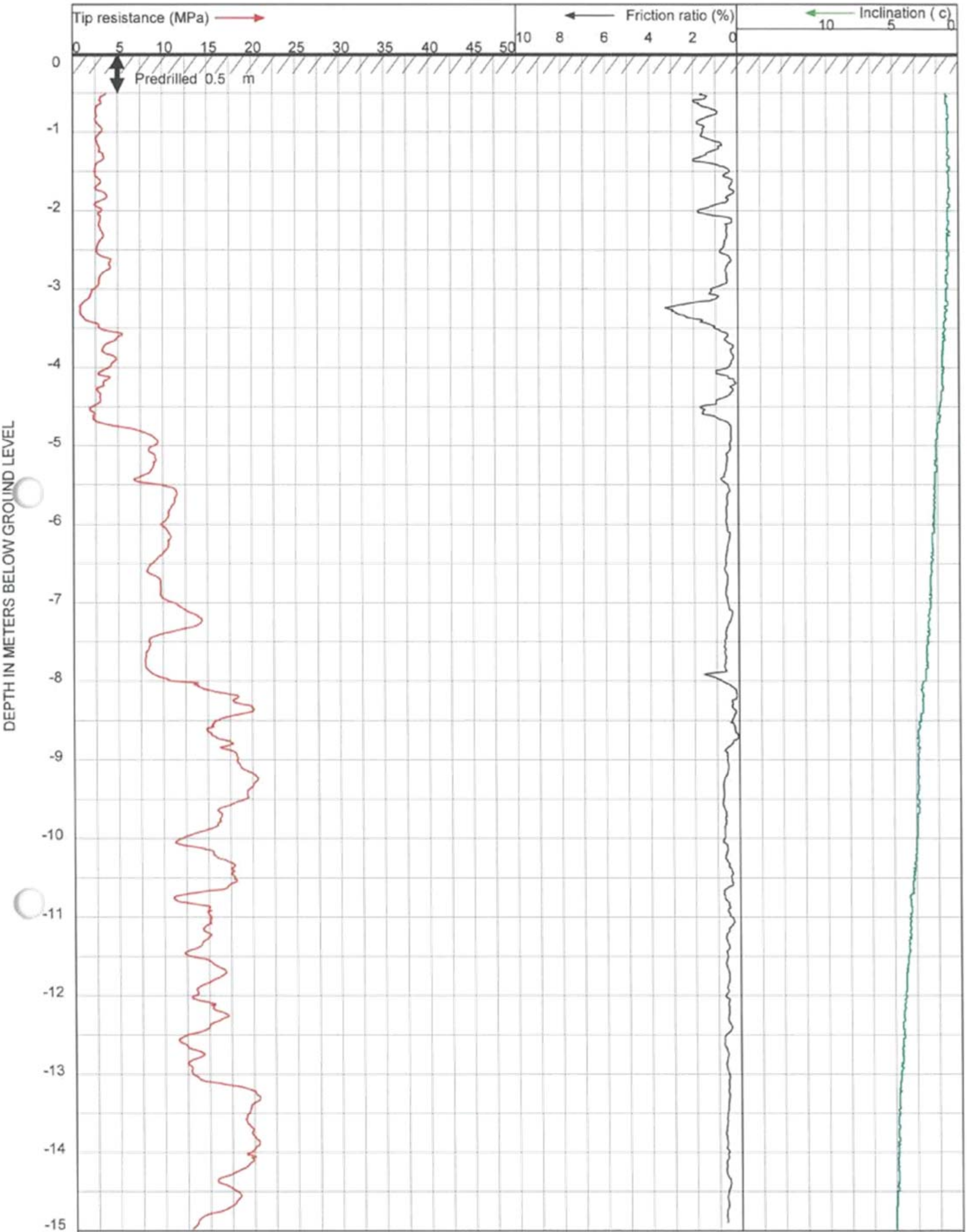
McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	9115	Date:	17-9-2011
CPT No:	CPT003	Operator:	D Keown
Project:	Aurecon Ltd	Remark:	Effective Refusal
Location:	27 Shirley Road, Christchurch		

McMILLAN
DRILLING SERVICES

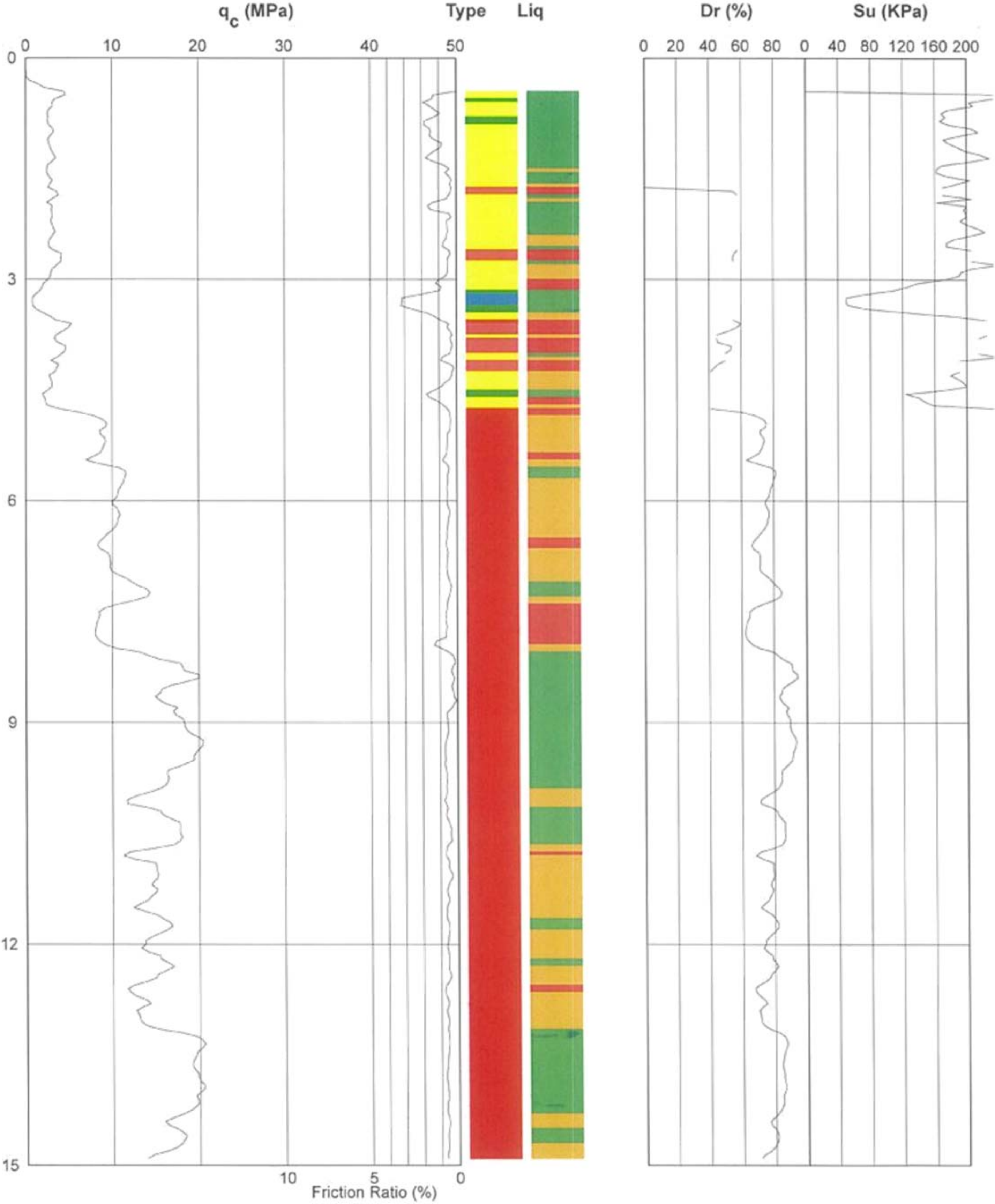


CLIENT : Aurecon Ltd
LOCATION : 27 Shirley Road, Christchurch
DATE : 17-9-2011
OPERATOR : D Keown
REMARK 1 : CPT004
REMARK 2 : Target depth

JOB # : 9115
TEST # : CPT004

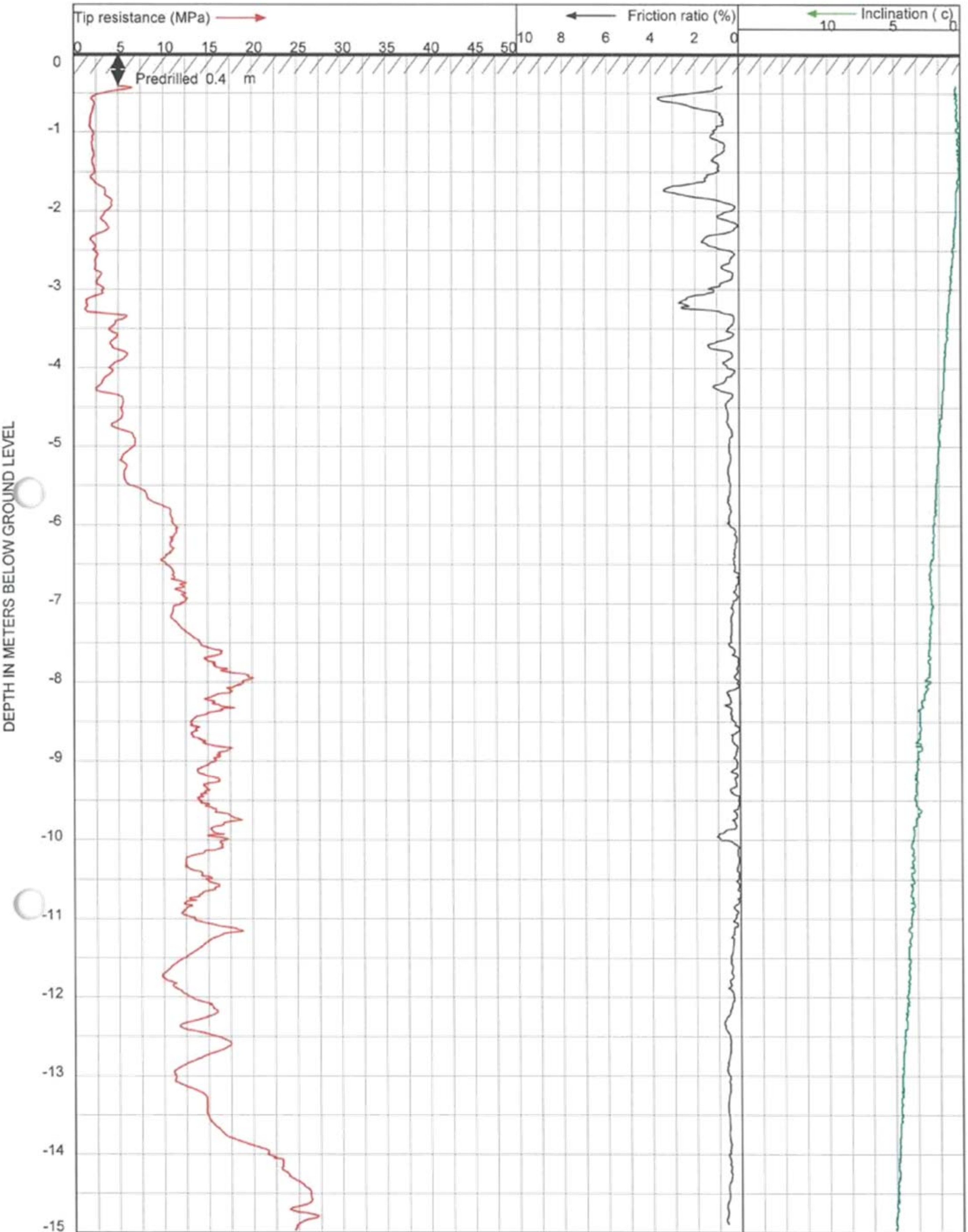
McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	9115	Date:	17-9-2011
CPT No:	CPT004	Operator:	D Keown
Project:	Aurecon Ltd	Remark:	Target depth
Location:	27 Shirley Road, Christchurch		

McMILLAN DRILLING SERVICES

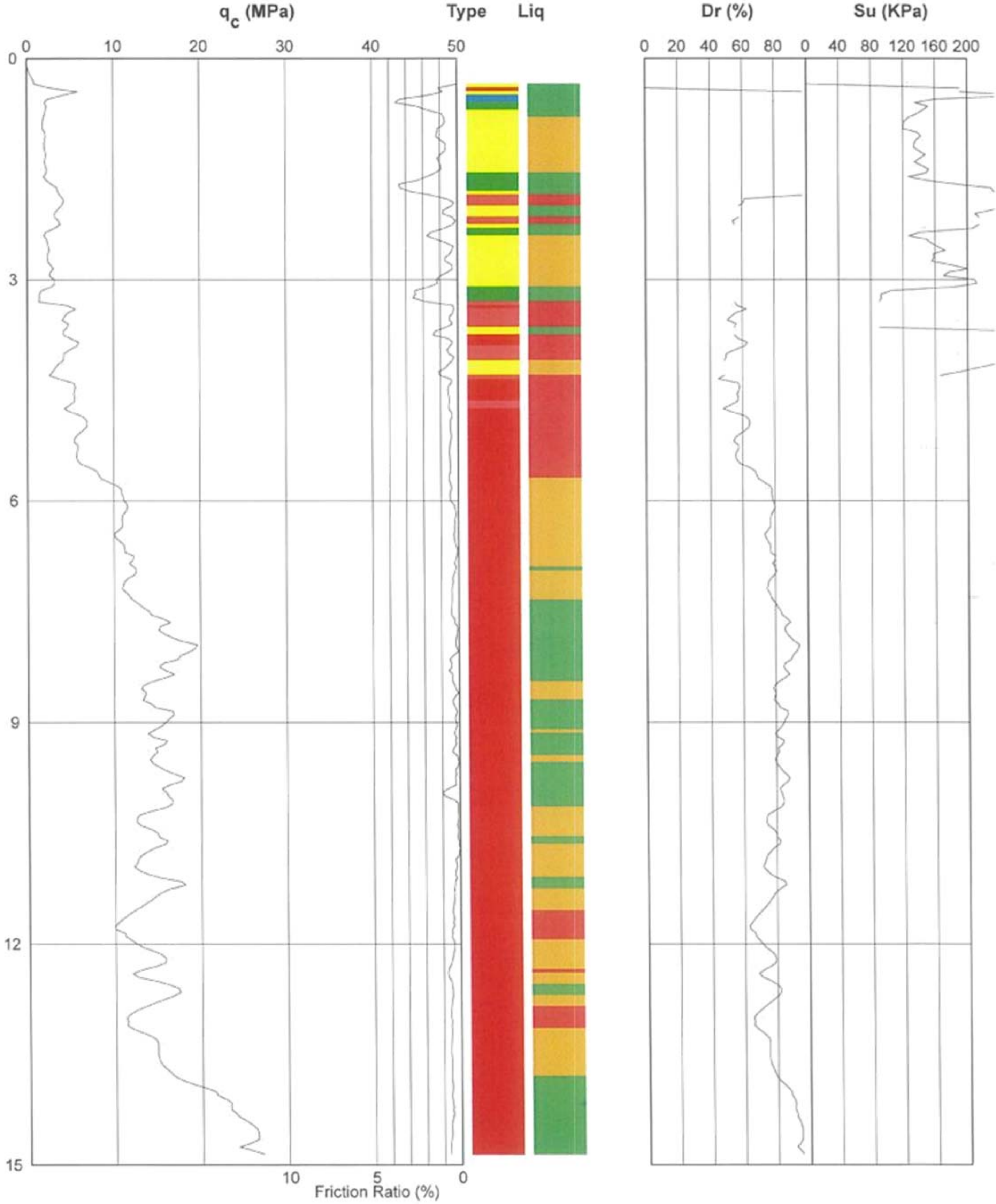


CLIENT : Aurecon Ltd
LOCATION : 27 Shirley Road, Christchurch
DATE : 17-9-2011
OPERATOR : D Keown
REMARK 1 : CPT005
REMARK 2 : Target Depth

JOB # : 9115
TEST #: CPT005

McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	9115	Date:	17-9-2011
CPT No:	CPT005	Operator:	D Keown
Project:	Aurecon Ltd	Remark:	Target Depth
Location:	27 Shirley Road, Christchurch		

McMILLAN DRILLING SERVICES

Appendix G – Research Rig Fact Sheets

RIG FACT SHEET

6x6 TRUCK RIG

DESCRIPTION

This CPT truck has been purpose built in the Netherlands by Geomil on a MAN 6 x 6 platform. It has 200 kN push rams with a push/pull clamp.

The truck is levelled using four hydraulically operated feet. It can operate on slopes of up to 15 degrees.

CAPABILITIES

The truck has a weight of 22 tons and has 20 ton push capacity.
The following operations can be performed:

- CPT/SCPT Testing
- DMT/SDMT Testing
- Mostap Sampling
- Piezometer Installation

The depth capacity is 60 m+ (depending on ground conditions.)

WEIGHTS AND DIMENSIONS

Height:	3.5 m (top down)
	4.4 m (top up)
Length:	7.4 m
Width:	2.5 m
Weight:	22,000 kg

GROUND INVESTIGATION
www.g-i.co.nz

IN SITU TESTING CPT - Seismic - DMT

AUCKLAND
133D Central Park Drive, Henderson, Auckland
PO Box 21 956 Henderson, Auckland 0650
Ph 09 950 1919

CHRISTCHURCH
38 Leeds Street
Phillipstown, Christchurch
Ph 03 928 1101

RIG FACT SHEET

PAGANI CPT RIG

DESCRIPTION

The Pagani TG63-150 is a fully hydraulic rig on rubber tracks which runs on a petrol engine. The compact size and light weight of the rig make it extremely manoeuvrable and able to access difficult sites. The rig is equipped with an automatic self-anchoring system, which quickly anchors the rig to the ground using 100 mm diameter continuous spiral screw anchors or larger diameter single spiral anchors.

The rig is transported to and from site in the back of a transit van or light truck. This makes the rig easy and quick to transport to site with low establishment cost. The light weight of the rig allows it to be easily lifted using a crane, Hiab or helicopter.

CAPABILITIES

With anchorage, the rig is able to push up to 15 tons. The anchors are very effective, giving good anchorage in almost all conditions (including soft clays and loose sands). The rig is also equipped with a drop hammer for dynamic testing and sampling. The drop hammer is a 63.5 kg hammer with a drop height of 750 mm (SPT configuration).

We can perform the following in situ tests with this rig:

• CPT/SCPT • DMT/SDMT • DPSH

In addition, continuous or targeted sampling can be undertaken with this rig. The maximum depth capacity for in situ testing is 30 m+.

WEIGHTS AND DIMENSIONS

Height:	1.5 m (mast down)
	3.8 m (mast up)
Length:	2.3 m
Width:	1.2 m
Weight:	1,100 kg

GROUND INVESTIGATION

www.g-i.co.nz

IN SITU TESTING CPT - Seismic - DMT

AUCKLAND
133D Central Park Drive, Henderson, Auckland
PO Box 21 956 Henderson, Auckland 0650
Ph 09 950 1919

CHRISTCHURCH
38 Leeds Street
Phillipstown, Christchurch
Ph 03 928 1101

Appendix H – Research Pre DSM CPTs

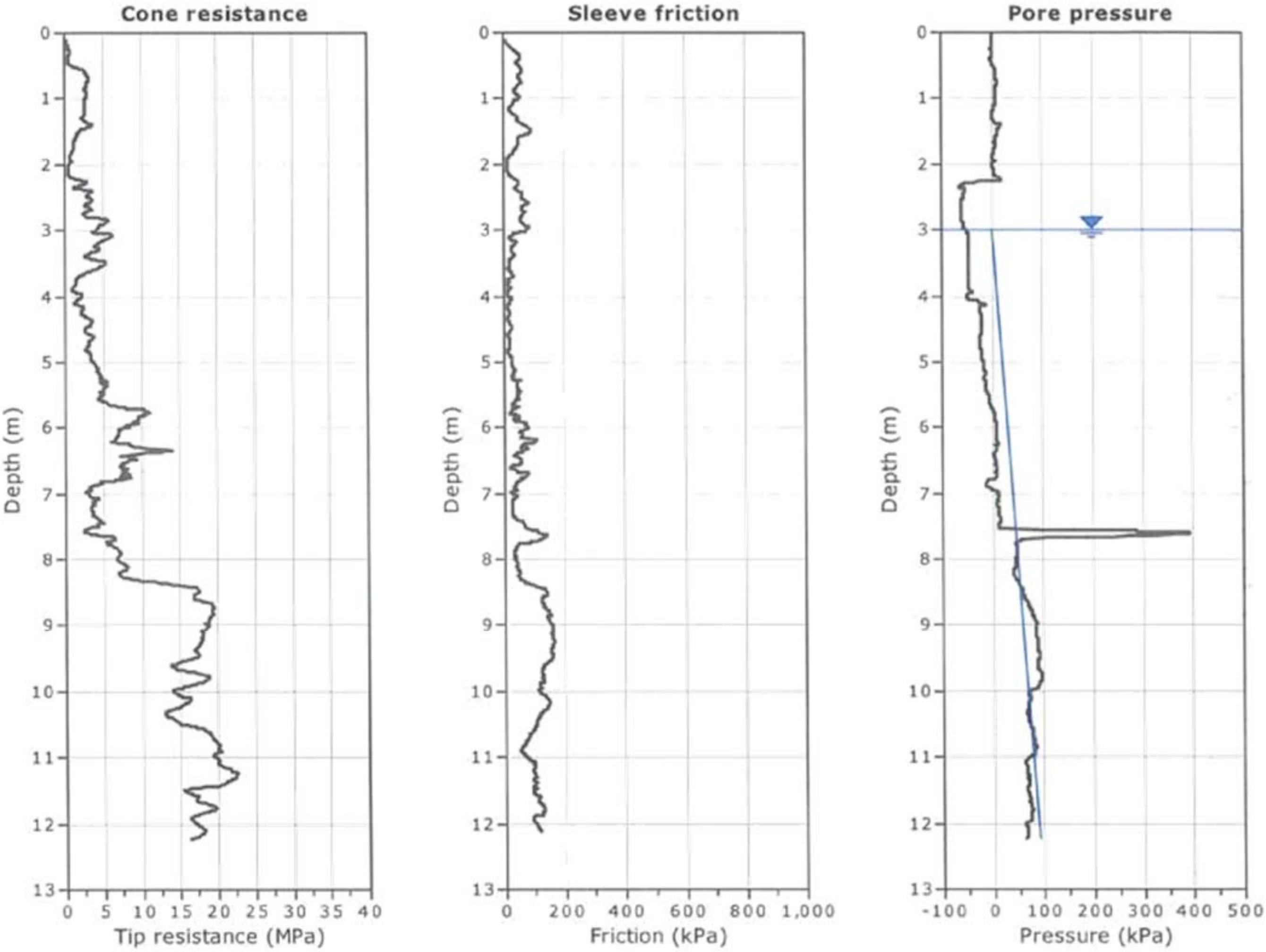
Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

CPT 7

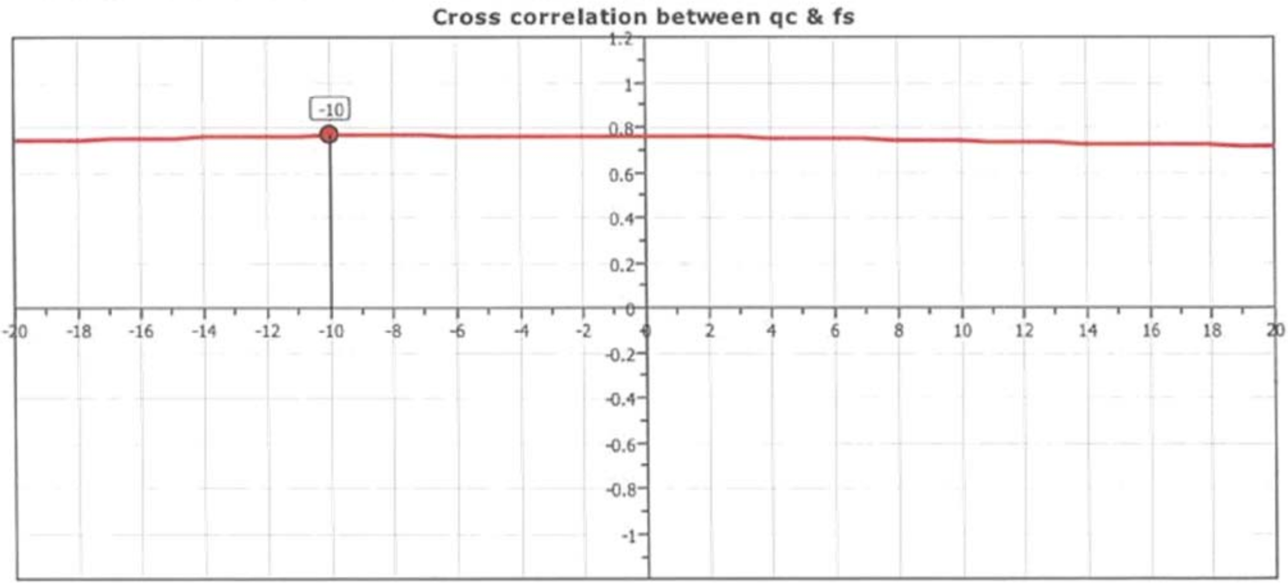
CPT: CPT-01

Total depth: 12.23 m, Date: 10/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch



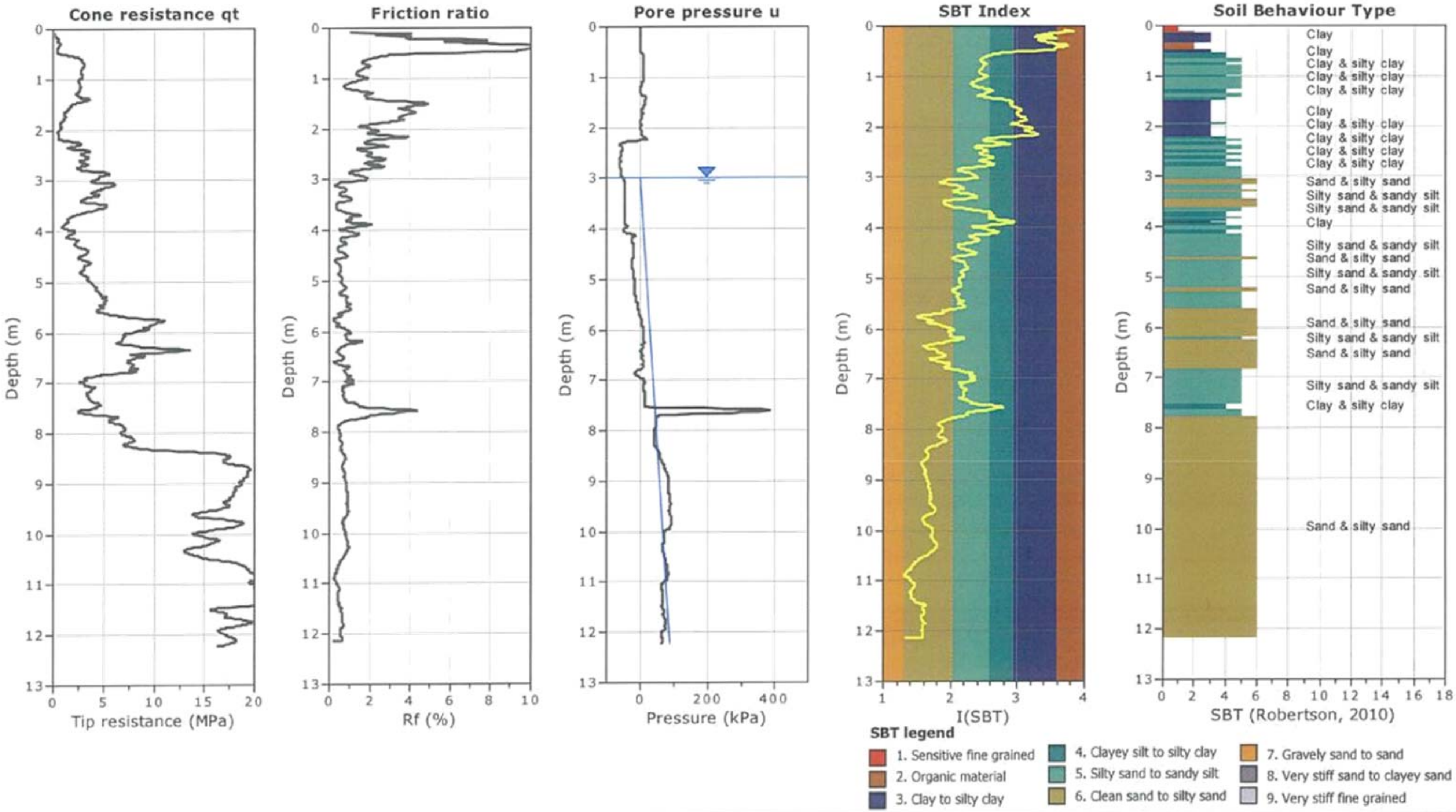
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

CPT: CPT-01
Total depth: 12.23 m, Date: 10/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE

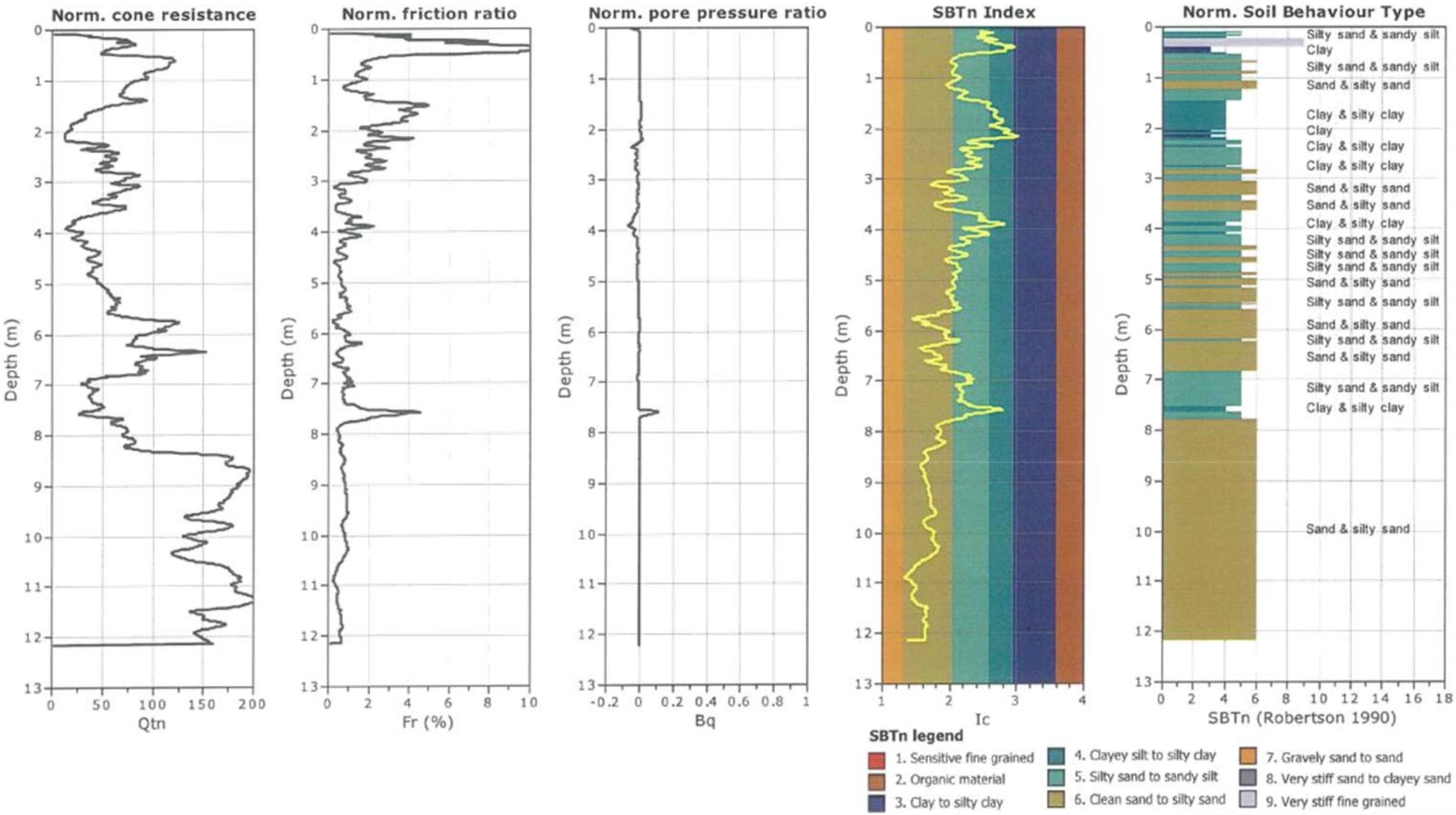


Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

CPT: CPT-01

Total depth: 12.23 m, Date: 10/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch



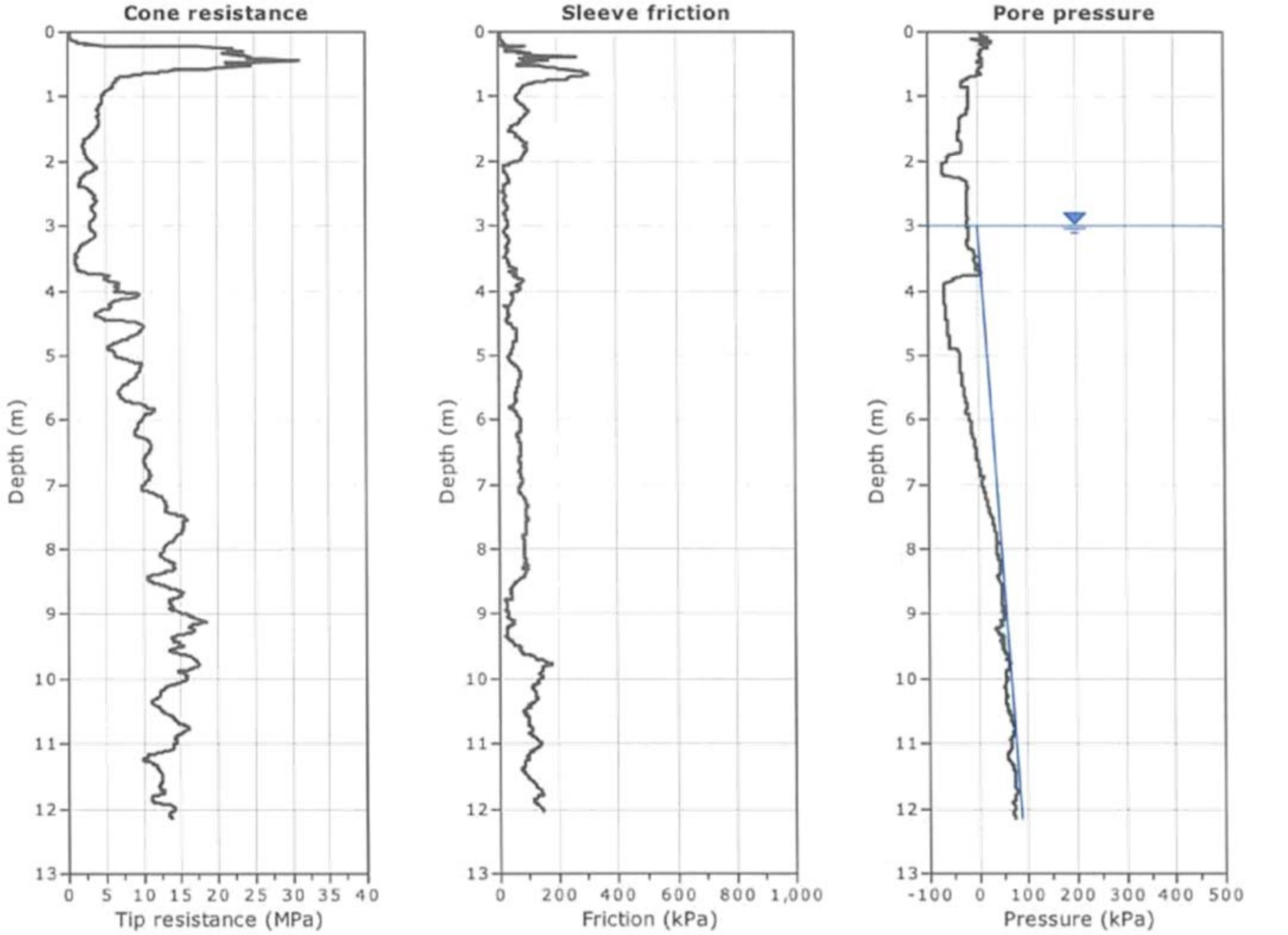
Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

CPT X

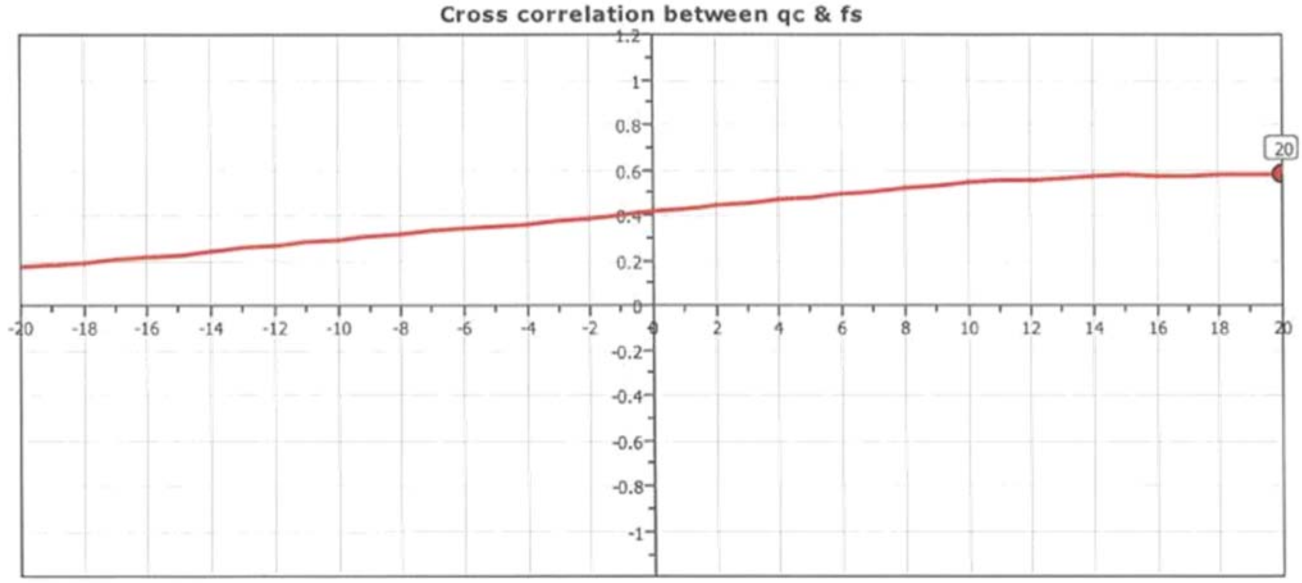
CPT: CPT-02

Total depth: 12.14 m, Date: 11/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch



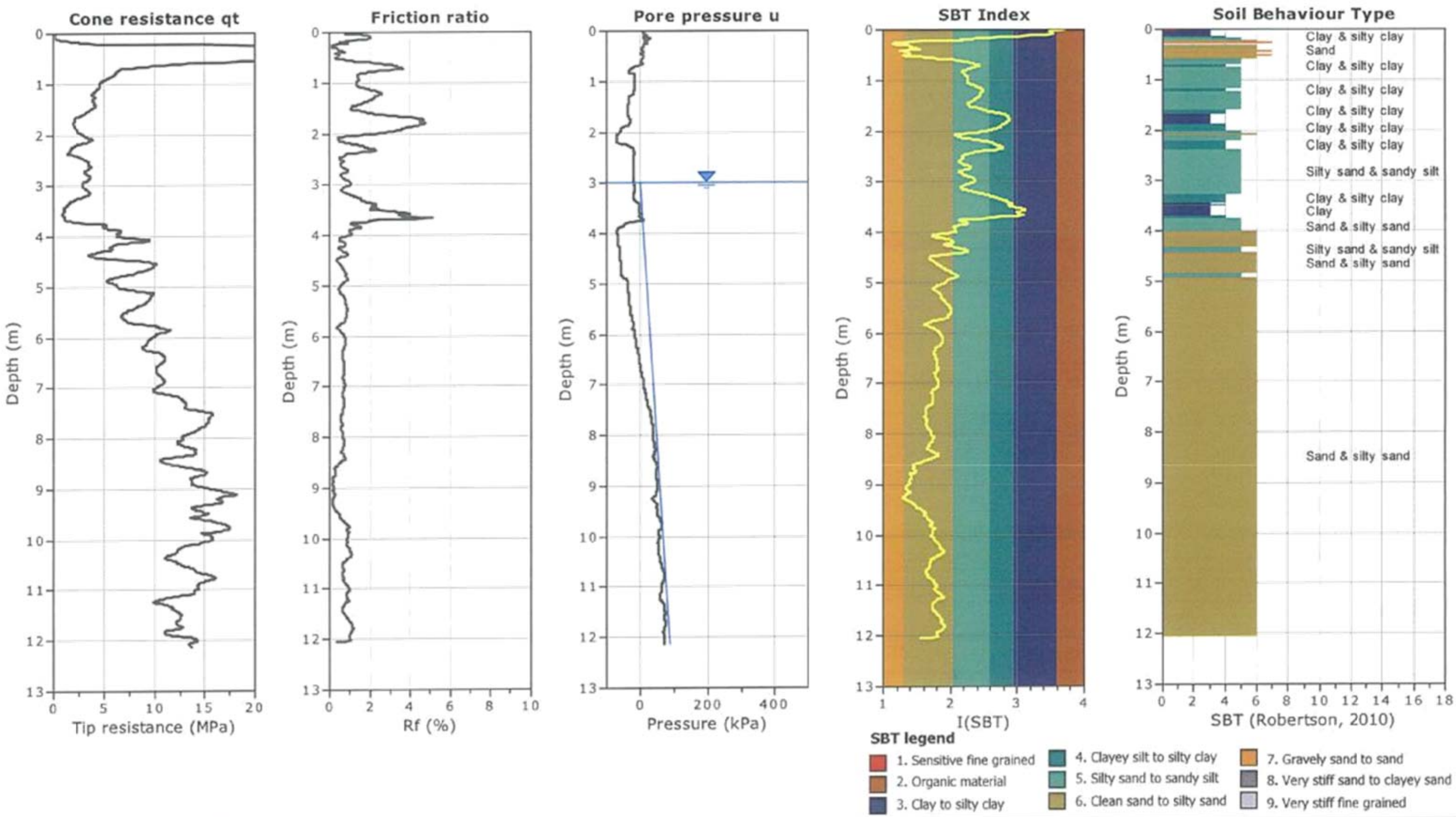
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

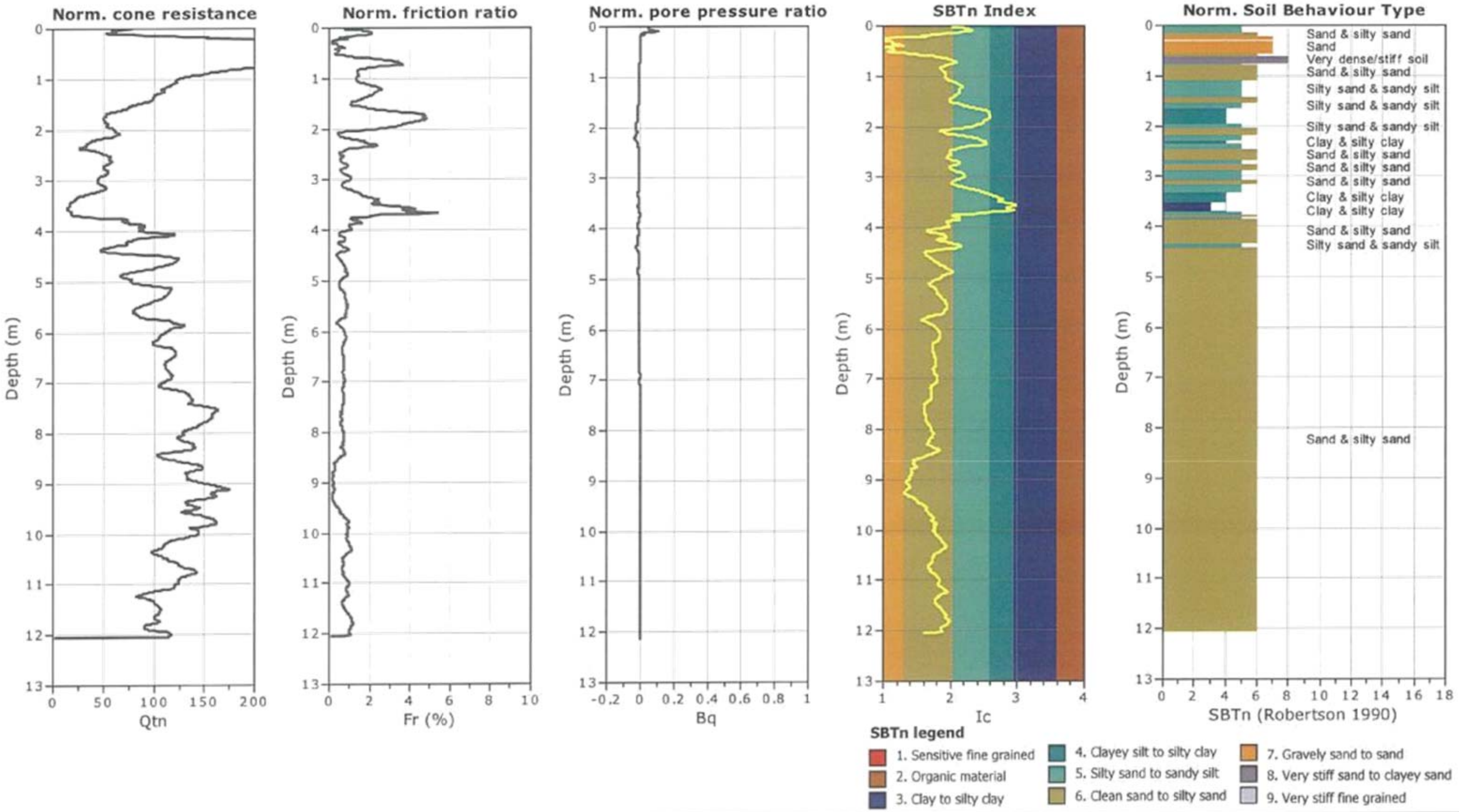
CPT: CPT-02
Total depth: 12.14 m, Date: 11/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE



Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

CPT: CPT-02
Total depth: 12.14 m, Date: 11/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE



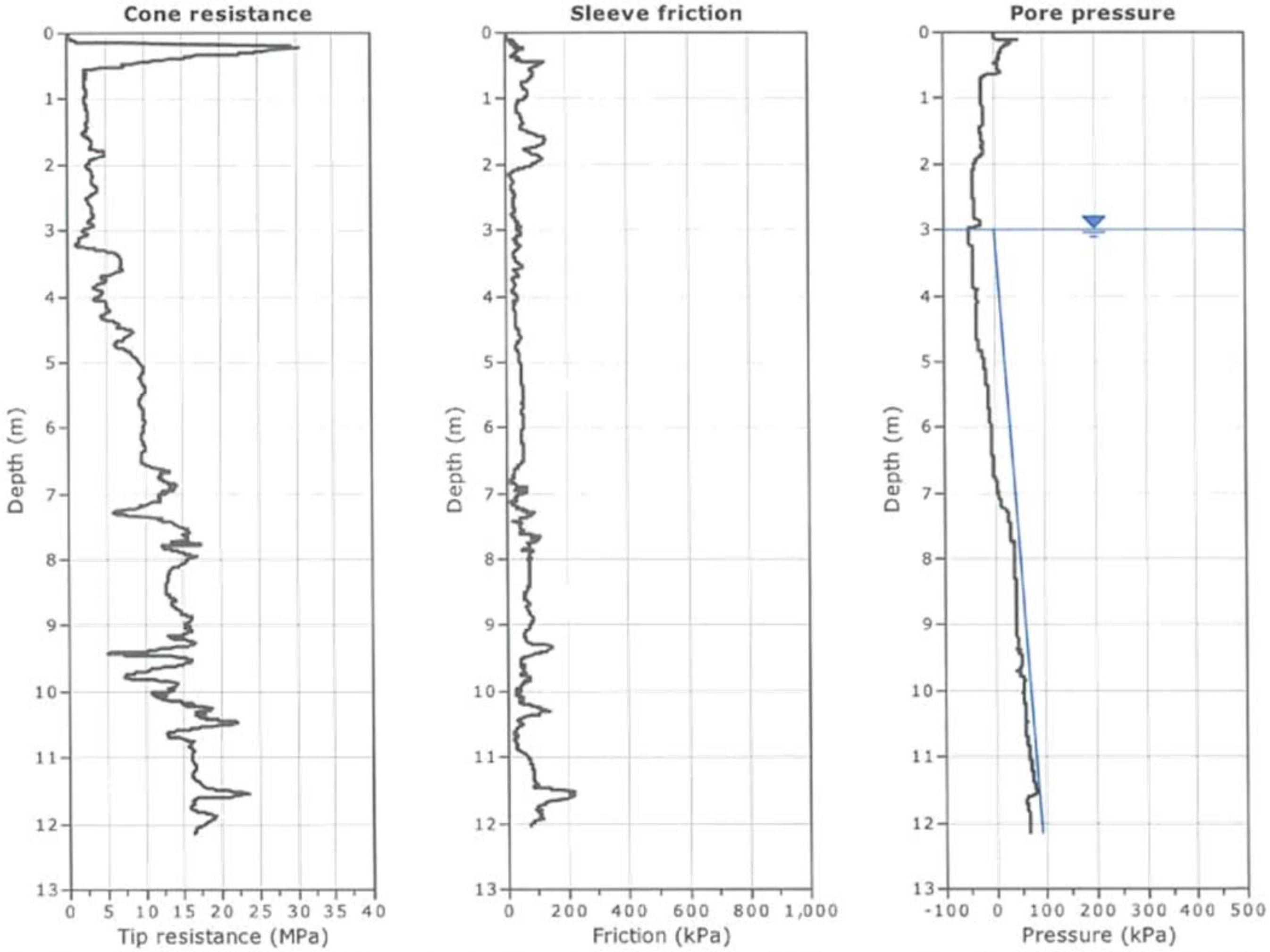
Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

CPT 4

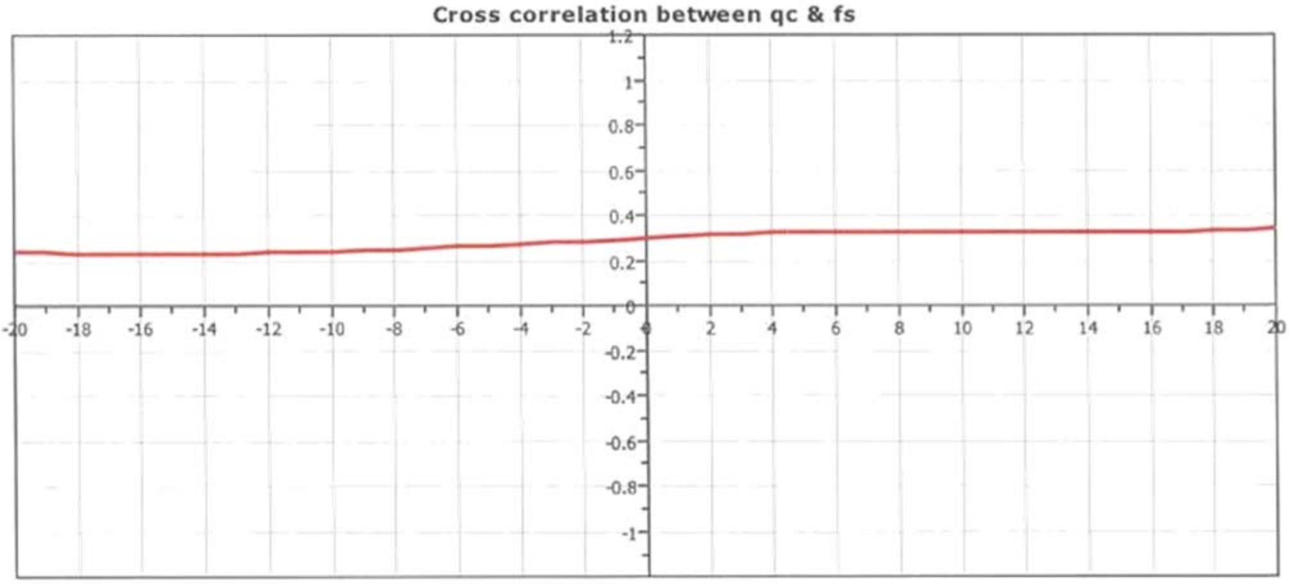
CPT: CPT-03

Total depth: 12.15 m, Date: 11/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch



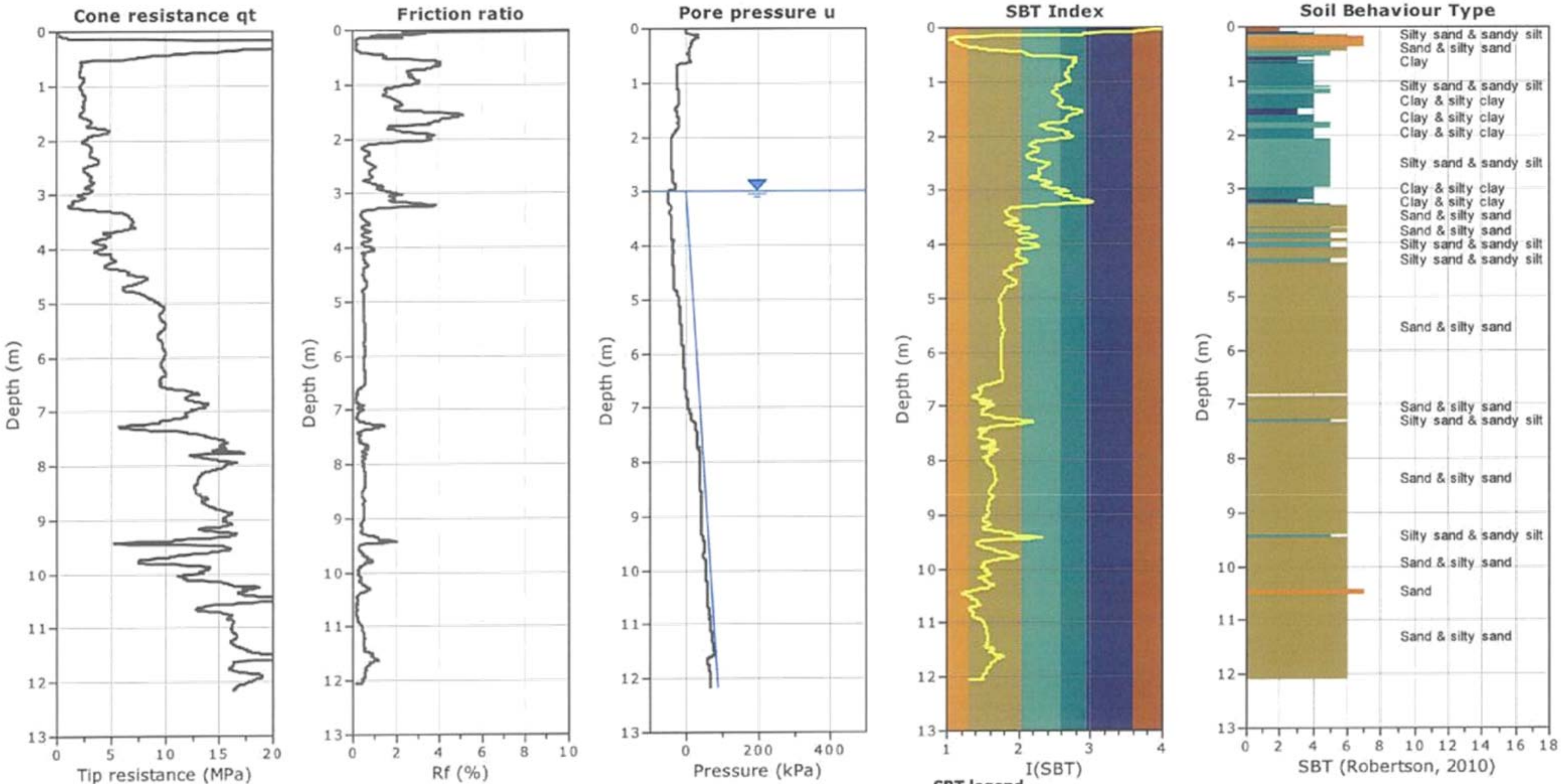
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

CPT: CPT-03
Total depth: 12.15 m, Date: 11/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE



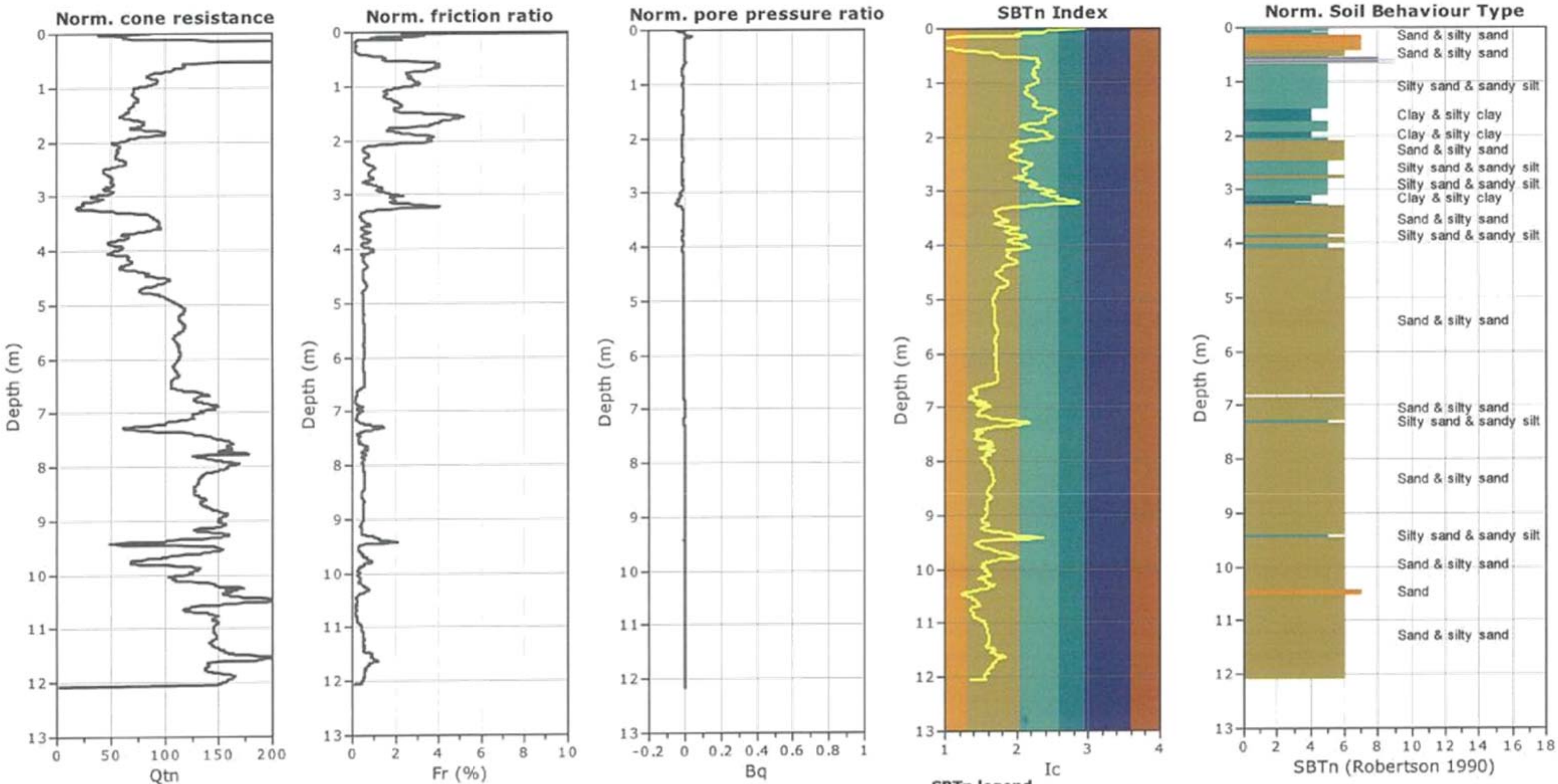
SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

CPT: CPT-03
Total depth: 12.15 m, Date: 11/09/2015
Cone Type: 10 cm2, 50 MPa, Pagani Piezocone
Cone Operator: PH&BE



SBTn legend

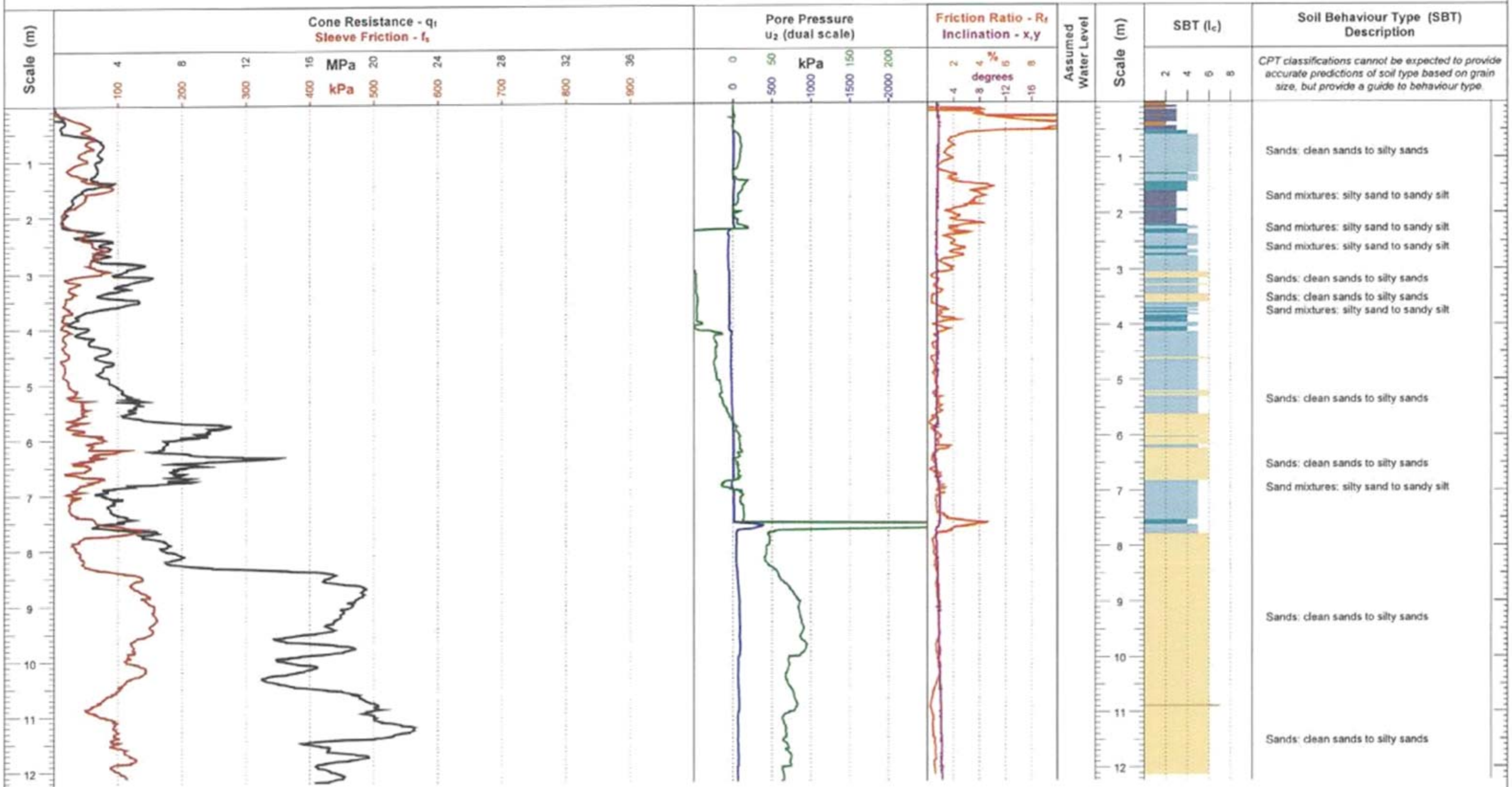
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Appendix I – Research Post DSM CPTs

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

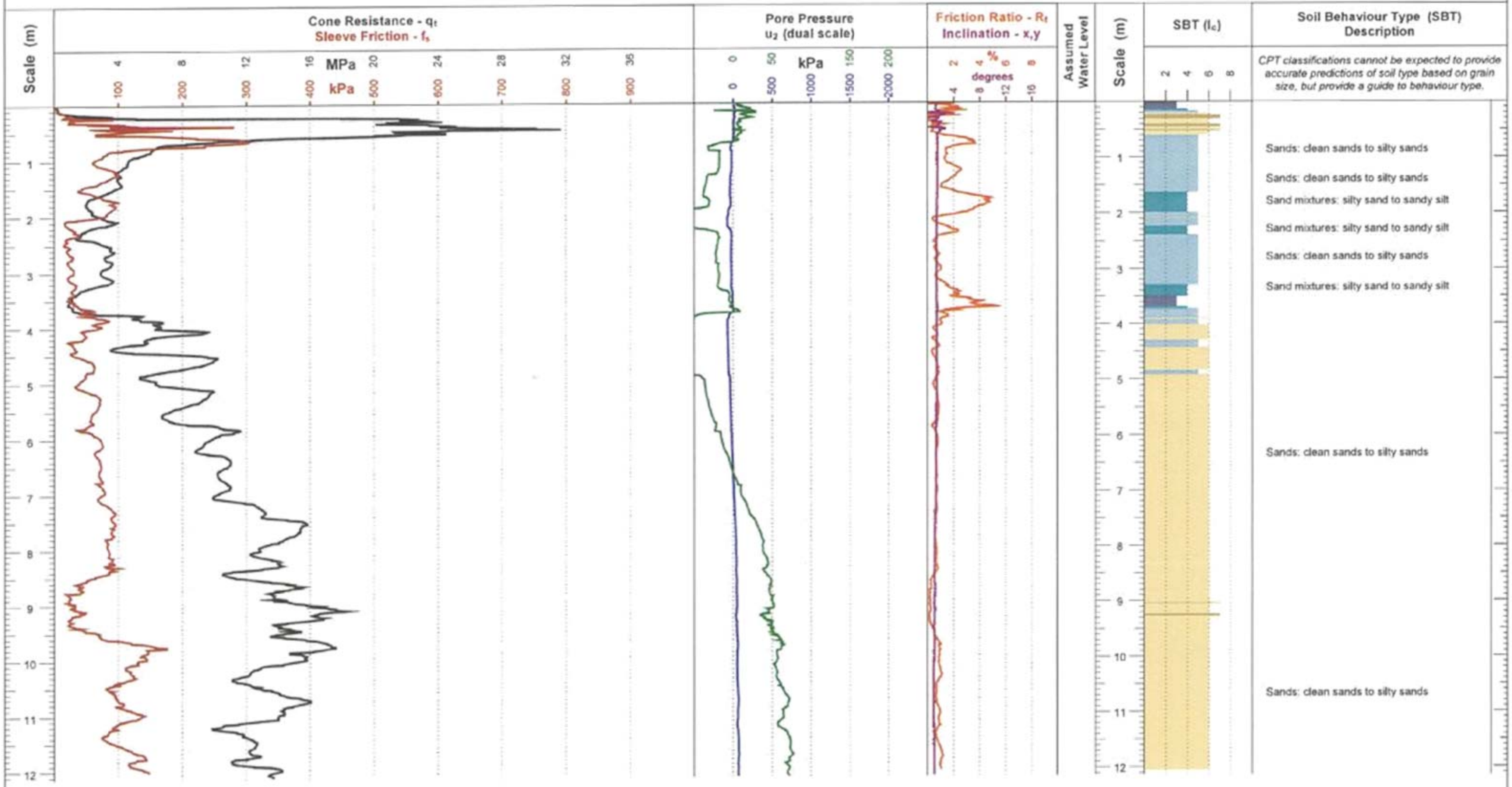


Client: Hiway Geotechnical	**North (m):** 5182809.53	**Remarks:**	**CPT Number:** CPT-01
Project Name: Willowlea Retirement Village	**East (m):** 1572246.97		
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -		
Project Engineer: Saad Ahmed	**Location Method:** Handheld GPS	**Operator:** Peter Haywood / **Cone Area Ratio:** 0.8	**Project ID:** 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.23	**Cone Number:** MKJ167 / **Sleeve Offset (m):** 0.1	**Date:** 10/09/2015 / Page 1 of 1

Hiway GeoTechnical

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION

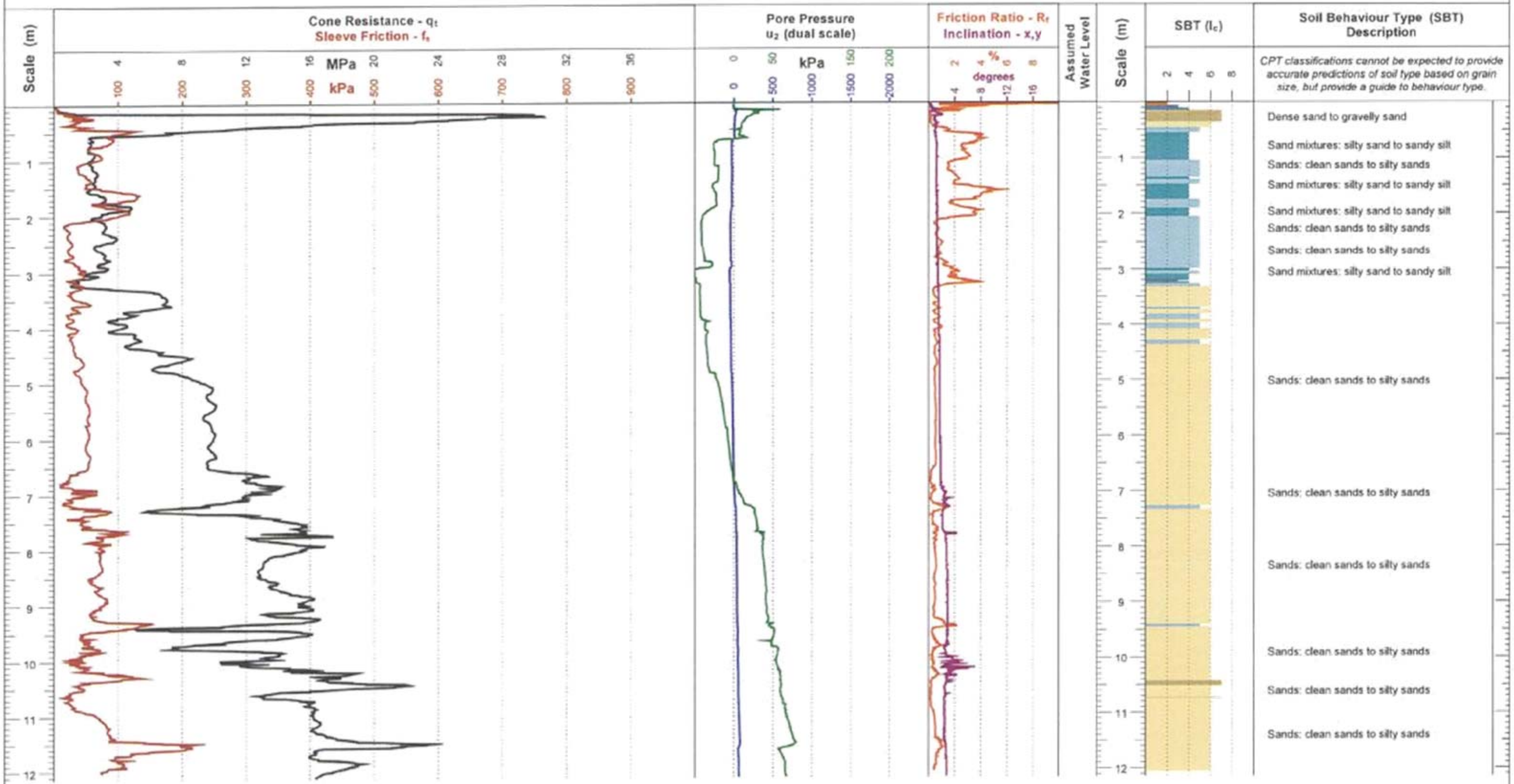


Client: Hiway Geotechnical	**North (m):** 5182764.20	**Remarks:**		**CPT Number:**	**CPT-02**
Project Name: Willowlea Retirement Village	**East (m):** 1572296.47				
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -				
Project Engineer: Saad Ahmed	**Location Method:** Handheld GPS	**Operator:** Peter Haywood	**Cone Area Ratio:** 0.8	**Project ID:**	**15-169**
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.14	**Cone Number:** MKJ291	**Sleeve Offset (m):** 0.1	**Date:** 11/09/2015	Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

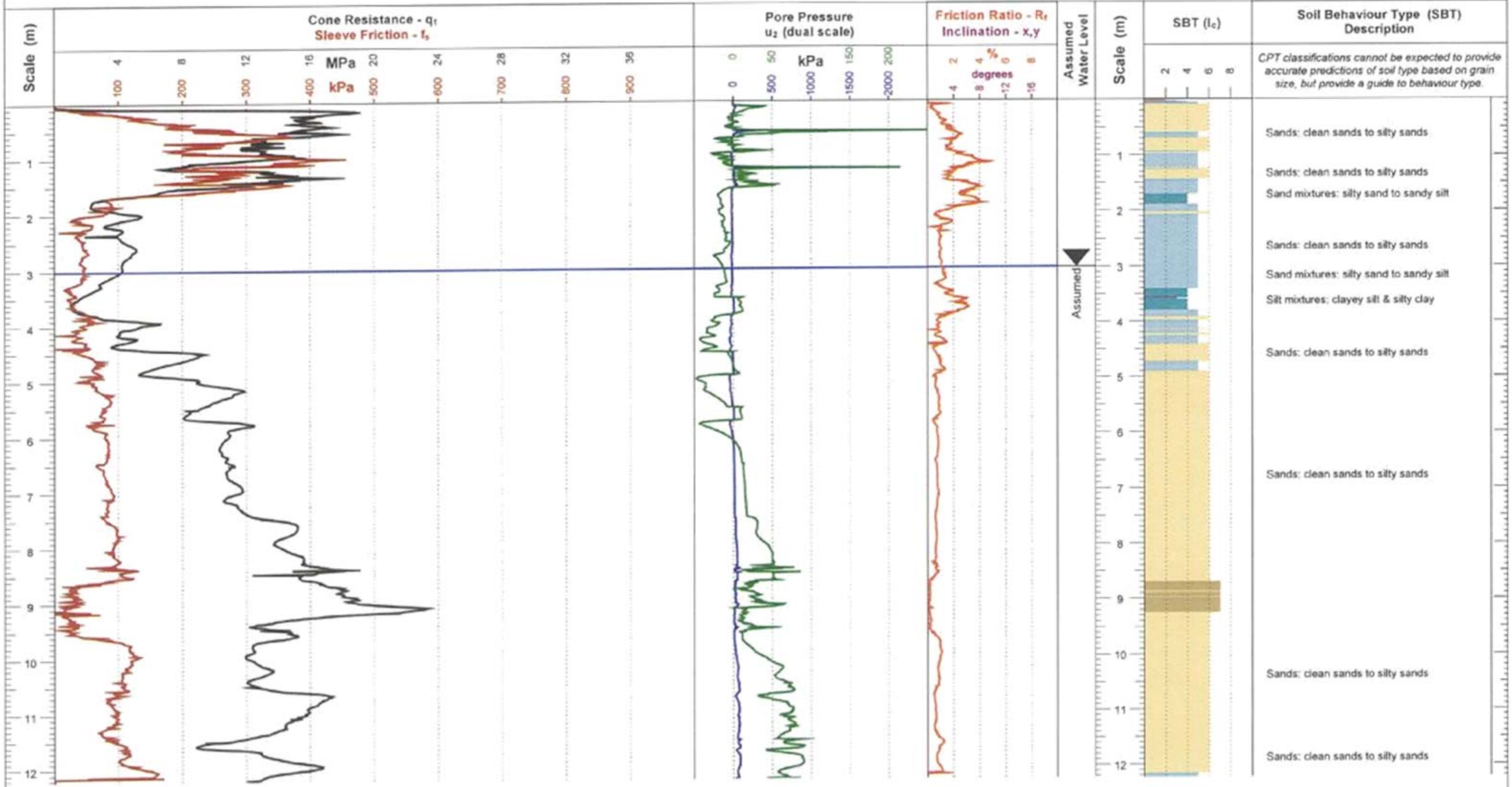


Client: Hiway Geotechnical	**North (m):** 5182721.91	**Remarks:**		**CPT Number:** CPT-03
Project Name: Willowlea Retirement Village	**East (m):** 1572274.82			
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -			
Project Engineer: Saad Ahmed	**Location Method:** Handheld GPS	**Operator:** Peter Haywood	**Cone Area Ratio:** 0.8	**Project ID:** 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.15	**Cone Number:** MKJ167	**Sleeve Offset (m):** 0.1	**Date:** 11/09/2015 Page 1 of 1

Hiway GeoTechnical

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION

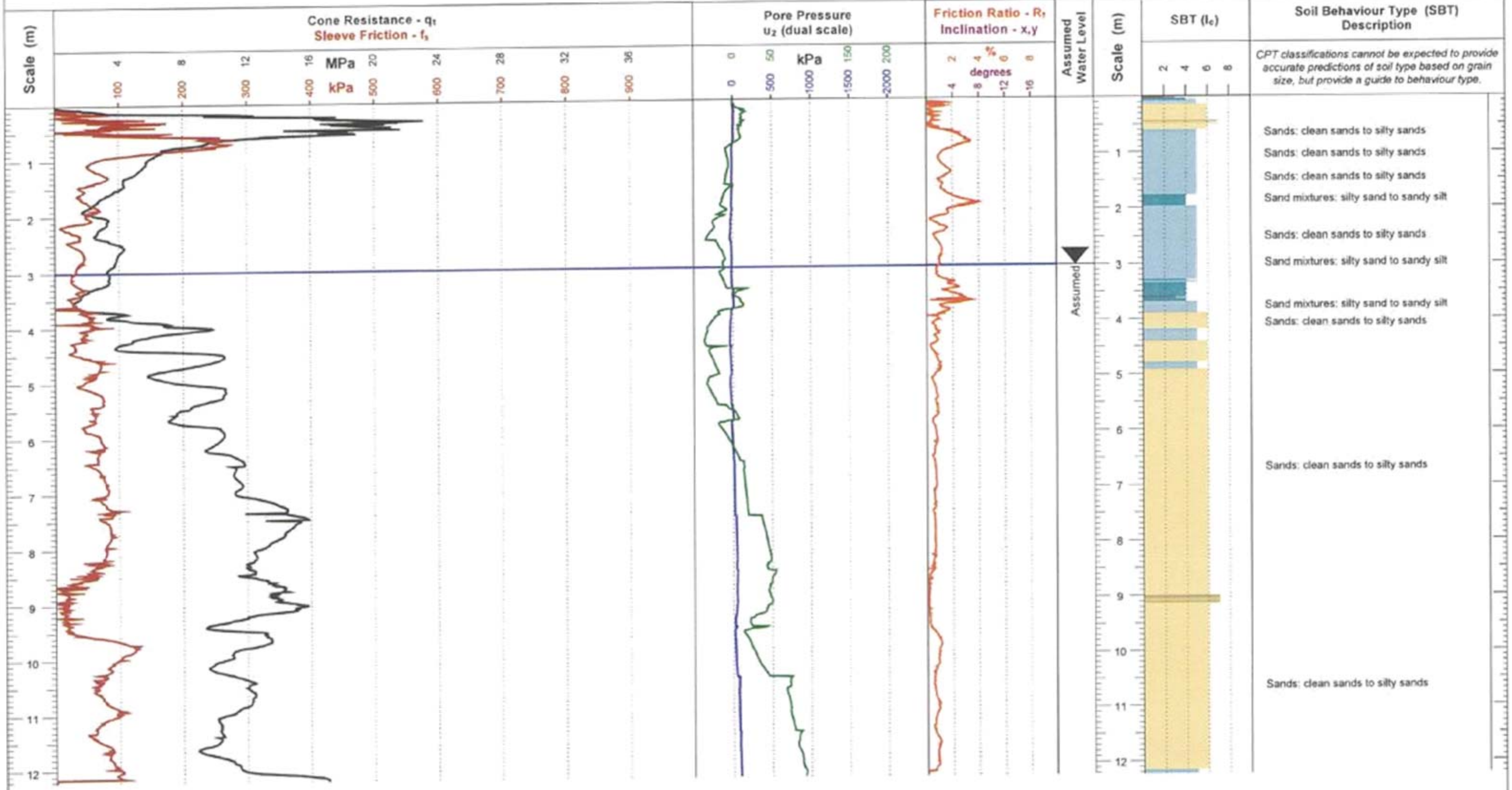


Client: Hiway Geotechnical	**North (m):** -	**Remarks:**		**CPT Number:**	**CPTX-500**
Project Name: Willowlea Retirement Village	**East (m):** -				
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -				
Project Engineer: Saad Ahmed	**Location Method:**	**Operator:** JC	**Cone Area Ratio:** 0.85	**Project ID:**	**15-169**
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.22	**Cone Number:** S15CFIIP.835	**Sleeve Offset (m):** 0	**Date:** 13/01/2016	Page 1 of 1

Hiway GeoTechnical

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: Hiway Geotechnical
Project Name: Willowlea Retirement Village
Location: 27 Shirley Road, Shirley, Christchurch
Project Engineer: Saad Ahmed
Contractor: Ground Investigation Ltd. www.g-i.co.nz

North (m): -
East (m): -
Elevation (m): -
Location Method:
Hole Depth (m): 12.22

Remarks:

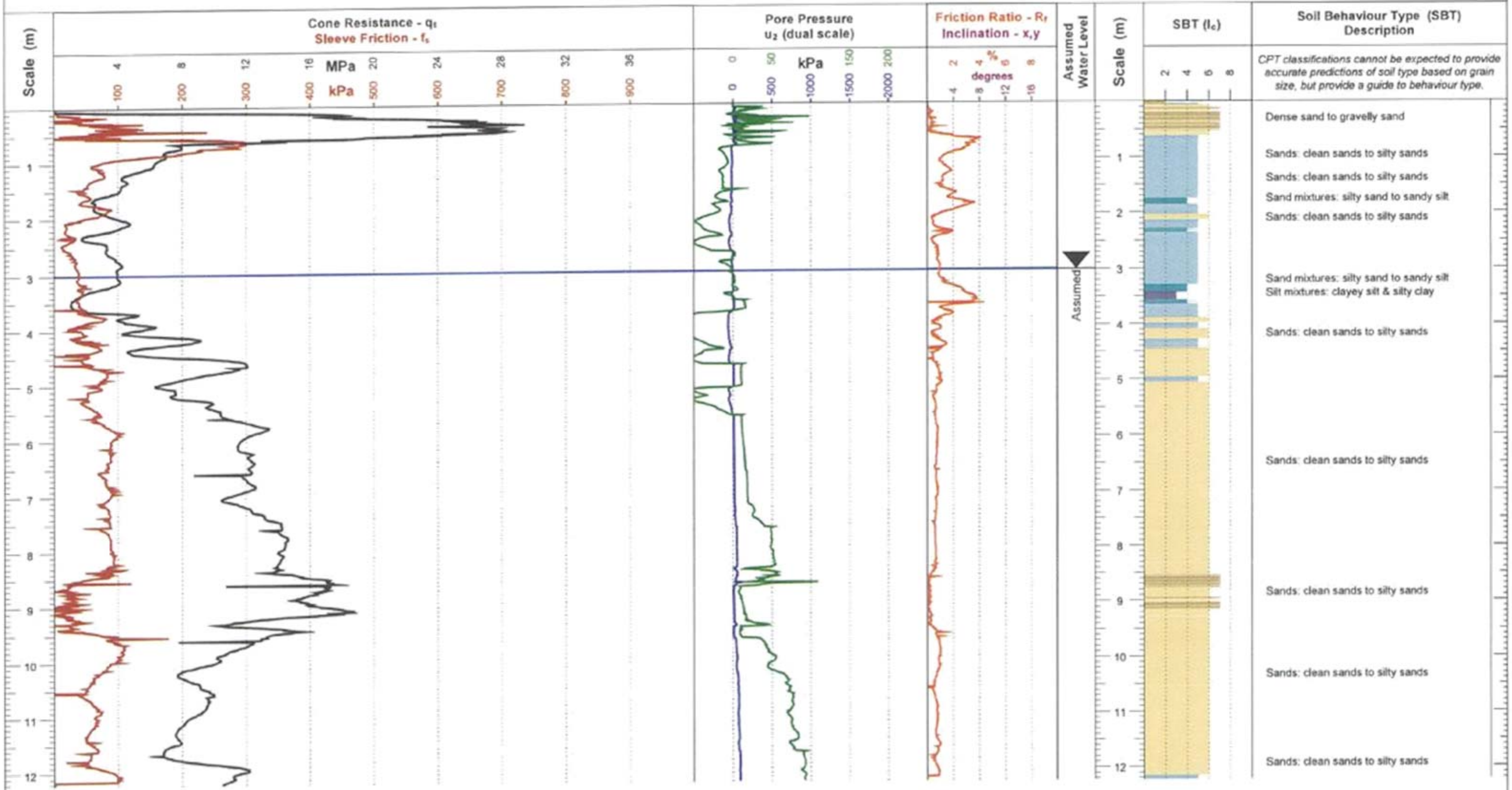
Operator: JC
Cone Number: S15CFIIP.835
Cone Area Ratio: 0.85
Sleeve Offset (m): 0

CPT Number: CPTX-750
Project ID: 15-169
Date: 13/01/2016
Page 1 of 1

Hiway GeoTechnical

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION

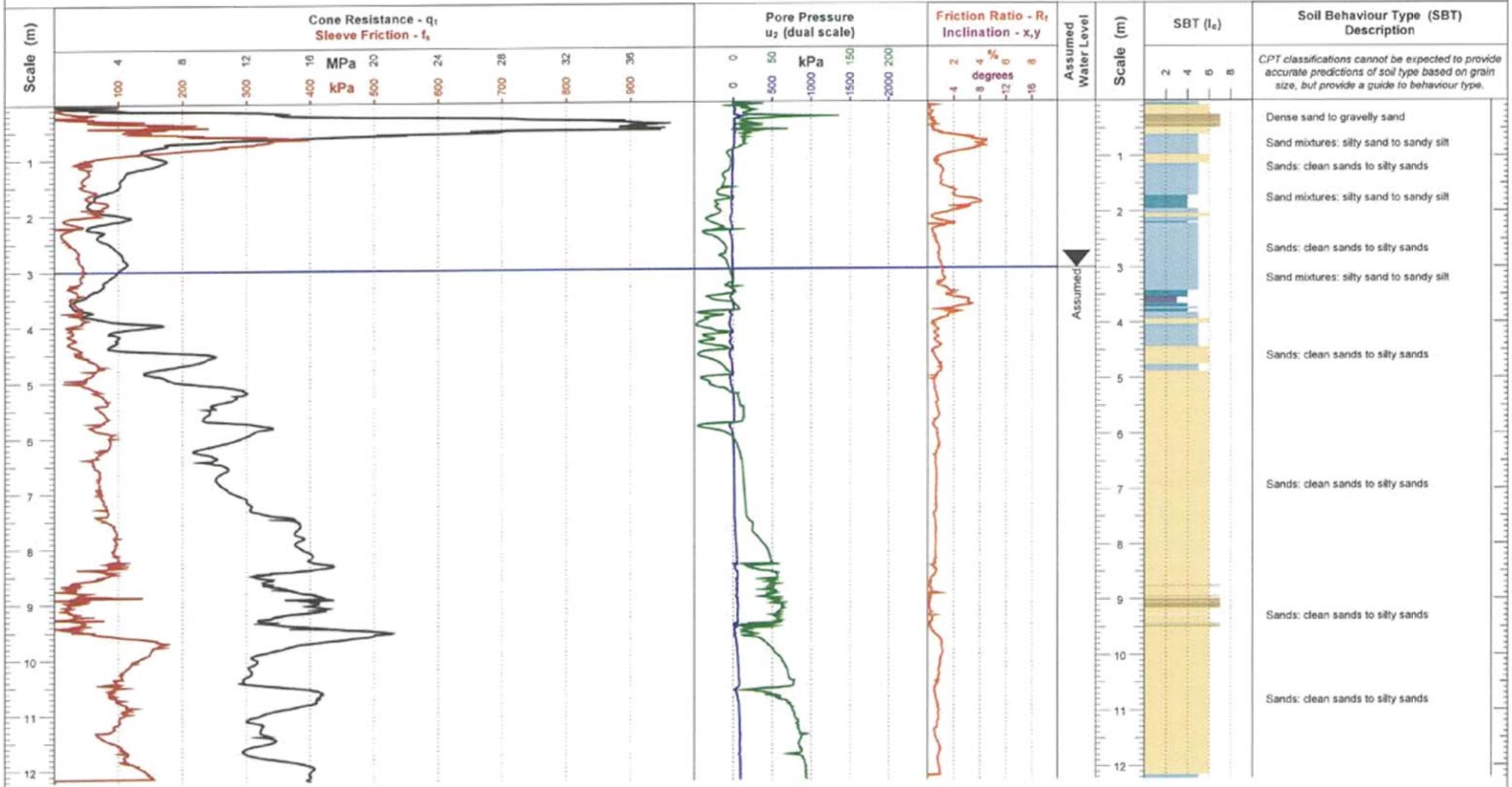


Client: Hiway Geotechnical	**North (m):** -	**Remarks:**		**CPT Number:** CPT-X1000
Project Name: Willowlea Retirement Village	**East (m):** -			
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -			
Project Engineer: Saad Ahmed	**Location Method:**	**Operator:** JC	**Cone Area Ratio:** 0.85	**Project ID:** 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.22	**Cone Number:** S15CFIIP.835	**Sleeve Offset (m):** 0	**Date:** 13/01/2016 Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

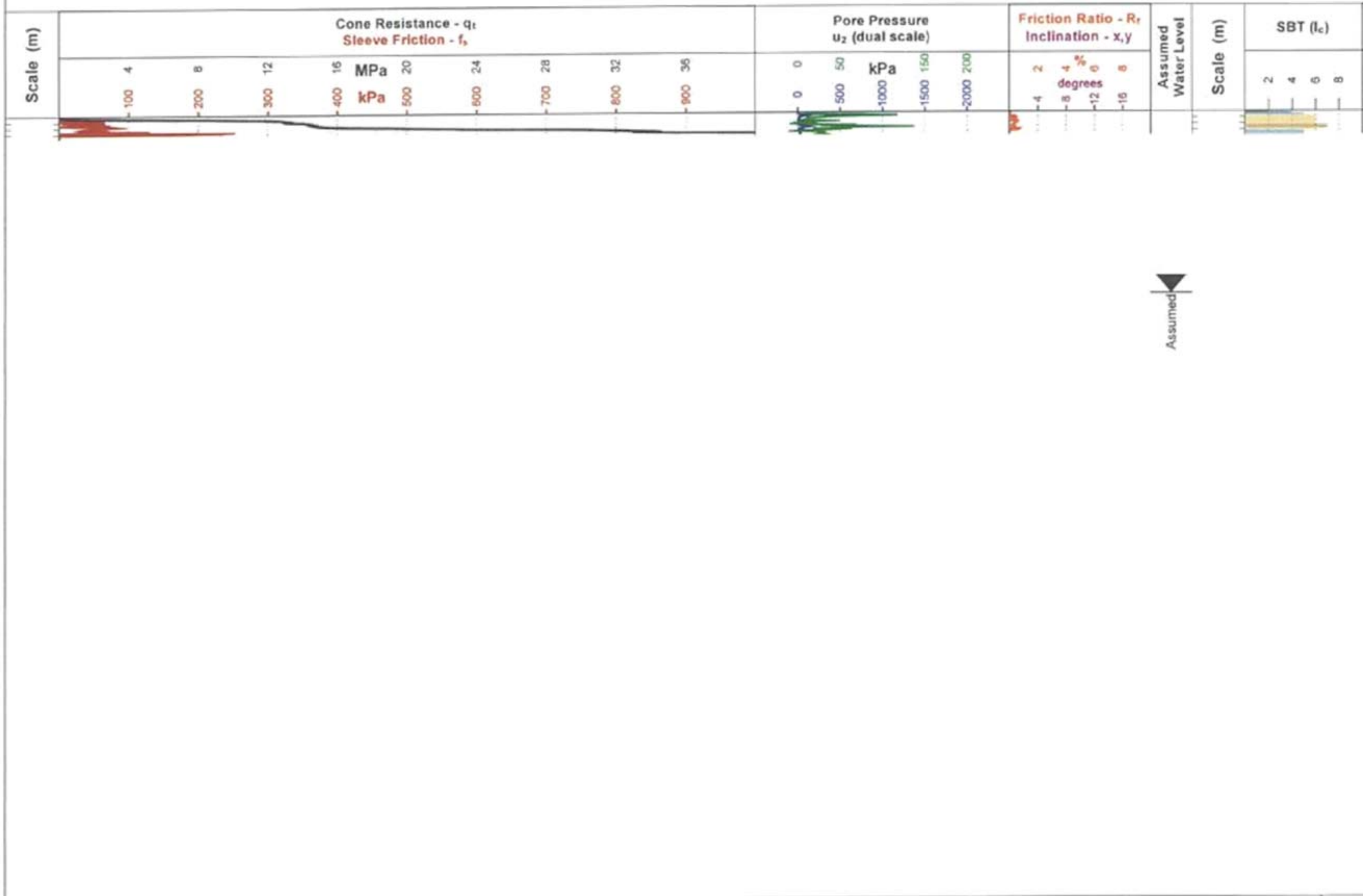


Client: Hiway Geotechnical	North (m): -	Remarks:		CPT Number: CPTX-1250
Project Name: Willowlea Retirement Village	East (m): -			
Location: 27 Shirley Road, Shirley, Christchurch	Elevation (m): -			
Project Engineer: Saad Ahmed	Location Method:	Operator: JC	Cone Area Ratio: 0.85	Project ID: 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	Hole Depth (m): 12.22	Cone Number: S15CFIIP.835	Sleeve Offset (m): 0	Date: 13/01/2016 — Page 1 of 1

Hiway GeoTechnical

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Soil Behaviour Type (SBT) Description

CPT classifications cannot be expected to provide accurate predictions of soil type based on grain size, but provide a guide to behaviour type.

Dense sand to gravelly sand

Client: Hiway Geotechnical
Project Name: Willowlea Retirement Village
Location: 27 Shirley Road, Shirley, Christchurch
Project Engineer: Saad Ahmed
Contractor: Ground Investigation Ltd. www.g-i.co.nz

North (m): -
East (m): -
Elevation (m): -
Location Method:
Hole Depth (m): 0.38

Remarks:

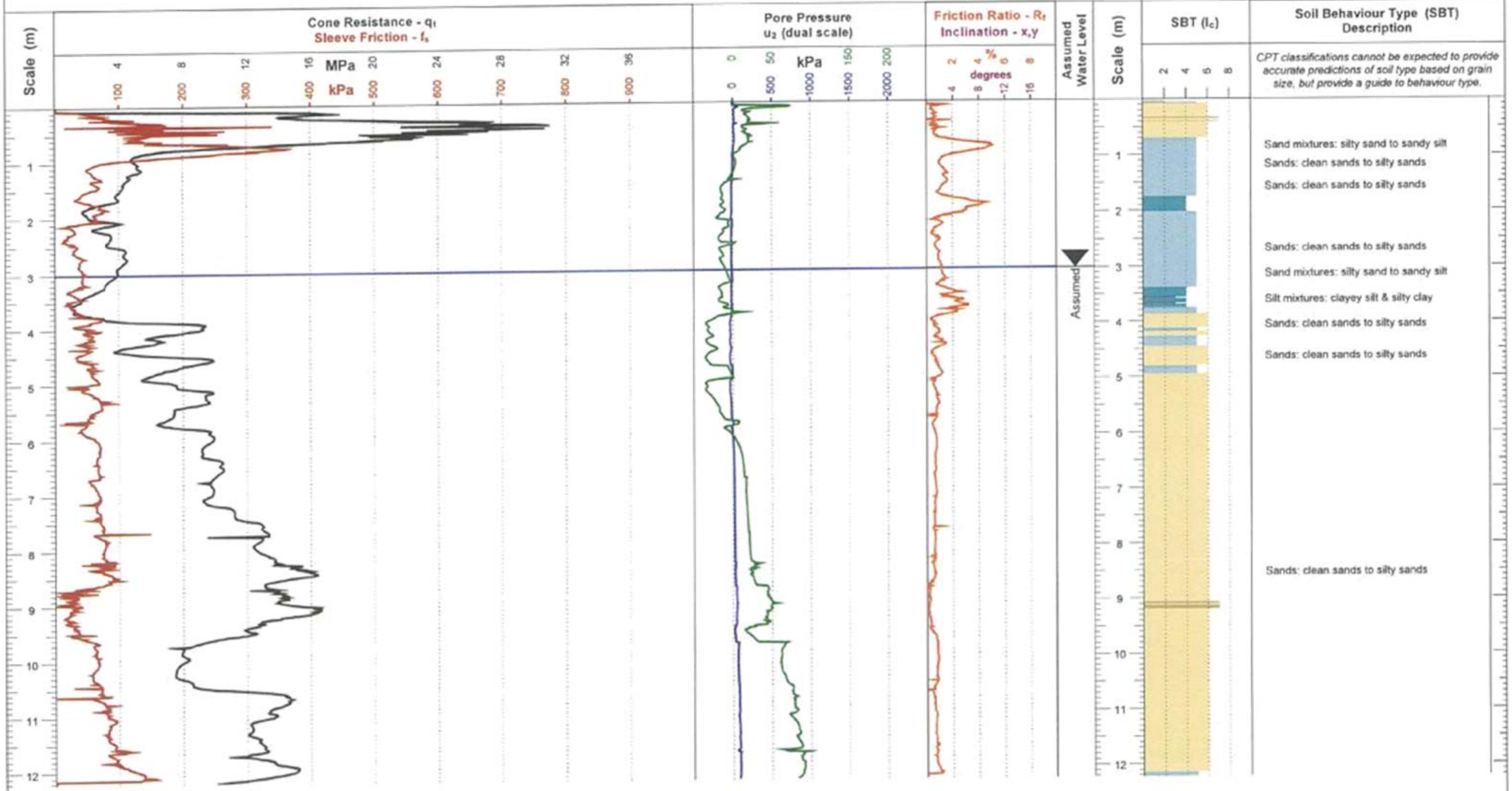
Operator: JC
Cone Number: S15CFIIP.835
Cone Area Ratio: 0.85
Sleeve Offset (m): 0

CPT Number: CPT-X1450
Project ID: 15-169
Date: 13/01/2016
Page 1 of 1

Hiway GeoTechnical

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION

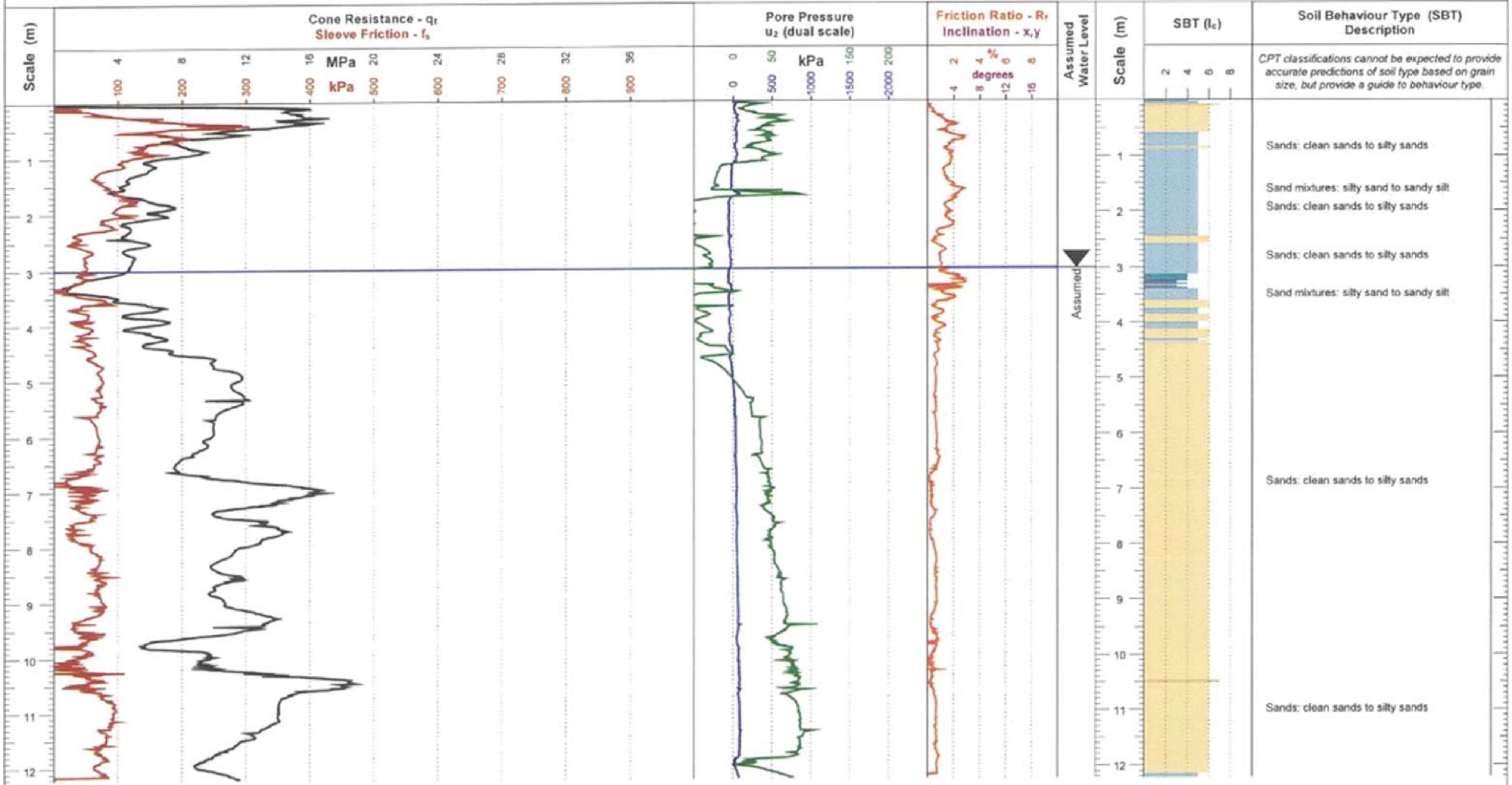


Client: Hiway Geotechnical	North (m): -	Remarks:	CPT Number: CPT-X1450A
Project Name: Willowlea Retirement Village	East (m): -		
Location: 27 Shirley Road, Shirley, Christchurch	Elevation (m): -		
Project Engineer: Saad Ahmed	Location Method:	Operator: JC / Cone Area Ratio: 0.85	Project ID: 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	Hole Depth (m): 12.21	Cone Number: S15CFIIP.835 / Sleeve Offset (m): 0	Date: 13/01/2016 / Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

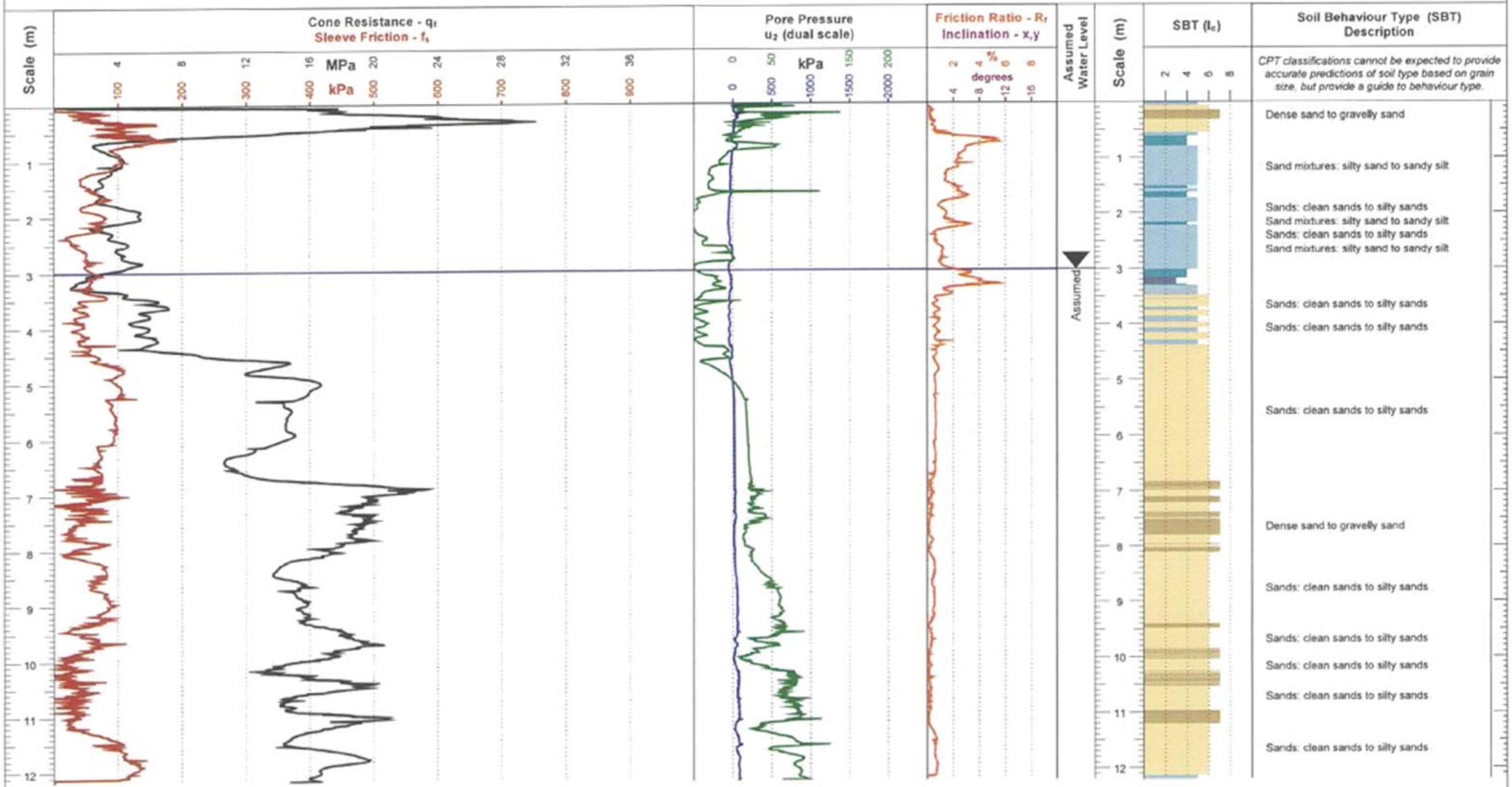


Client: Hiway Geotechnical	**North (m):** -	**Remarks:**		**CPT Number:** CPT-Y500
Project Name: Willowlea Retirement Village	**East (m):** -			
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -			
Project Engineer: Saad Ahmed	**Location Method:**	**Operator:** JC	**Cone Area Ratio:** 0.85	**Project ID:** 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.21	**Cone Number:** S15CFIIP.835	**Sleeve Offset (m):** 0	**Date:** 12/01/2016 Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION



Client: Hiway Geotechnical
Project Name: Willowlea Retirement Village
Location: 27 Shirley Road, Shirley, Christchurch
Project Engineer: Saad Ahmed
Contractor: Ground Investigation Ltd. www.g-i.co.nz

North (m): -
East (m): -
Elevation (m): -
Location Method:
Hole Depth (m): 12.20

Remarks:

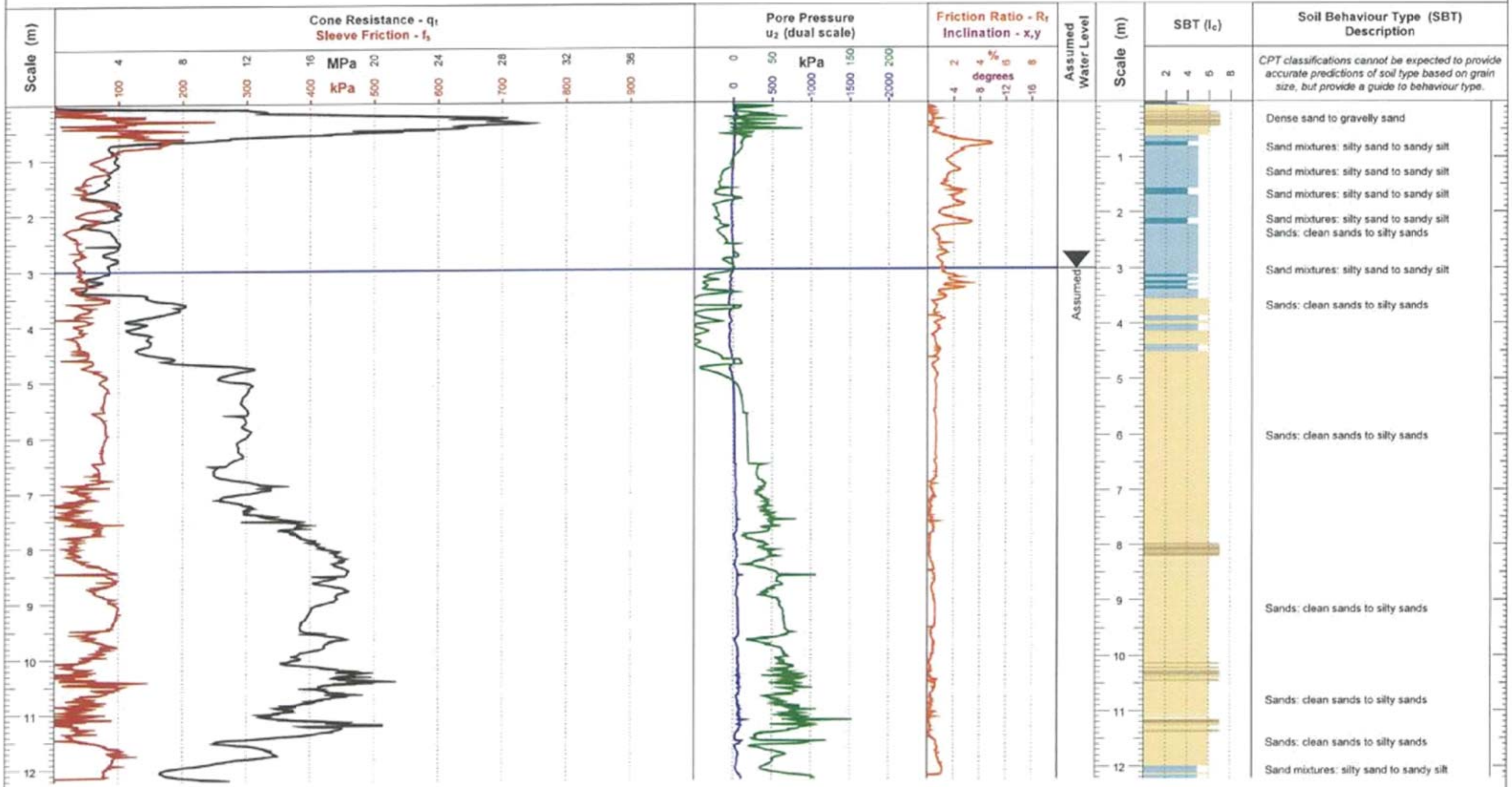
Operator: JC
Cone Number: S15CFIIP.835
Cone Area Ratio: 0.85
Sleeve Offset (m): 0

CPT Number: CPTY-750
Project ID: 15-169
Date: 12/01/2016
Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

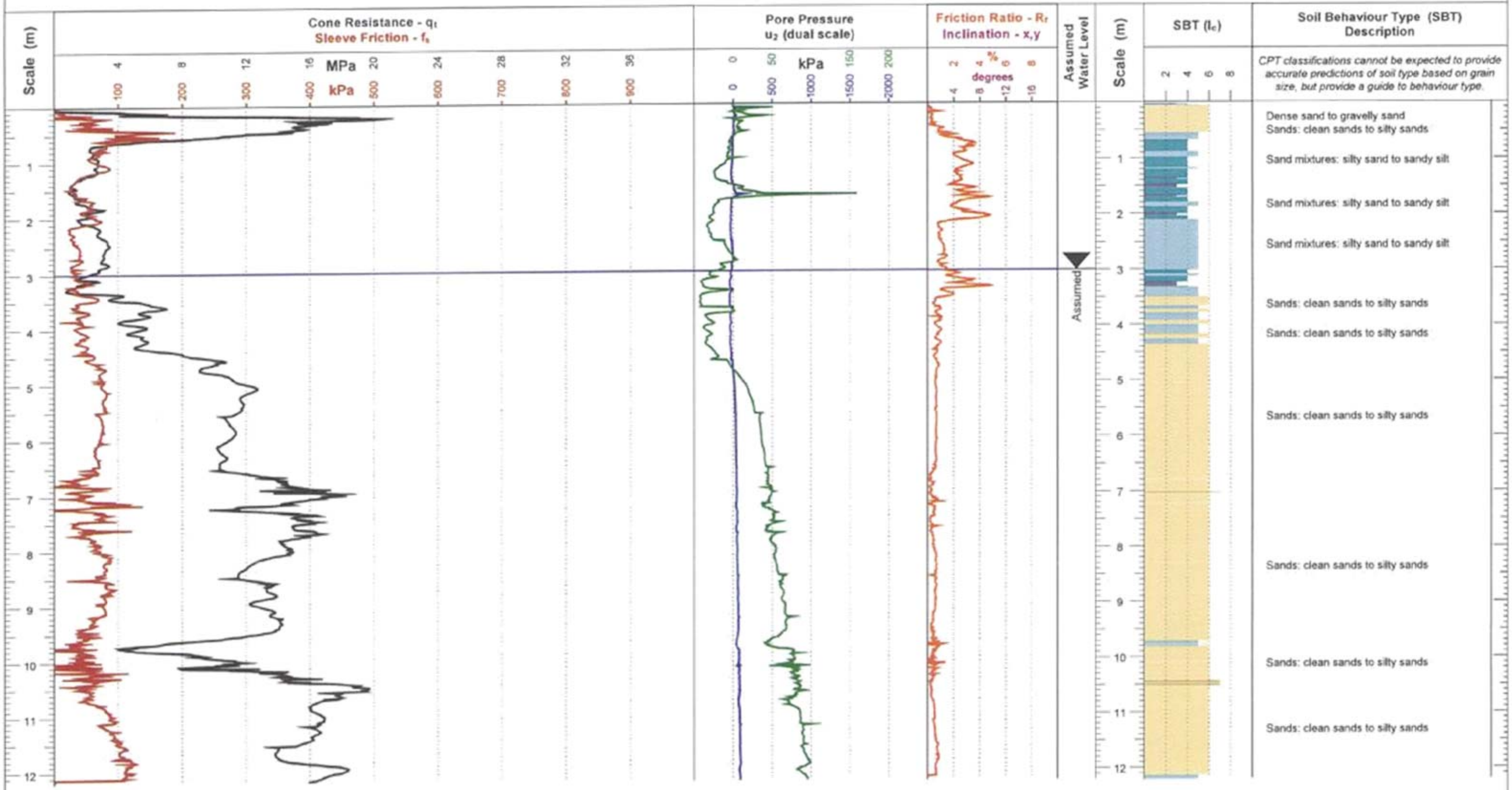


Client: Hiway Geotechnical	North (m): -	Remarks:	CPT Number: CPT-Y1000
Project Name: Willowlea Retirement Village	East (m): -		
Location: 27 Shirley Road, Shirley, Christchurch	Elevation (m): -		
Project Engineer: Saad Ahmed	Location Method:	Operator: JC / Cone Area Ratio: 0.85	Project ID: 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	Hole Depth (m): 12.21	Cone Number: S15CFIIP.835 / Sleeve Offset (m): 0	Date: 1/01/2003 / Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

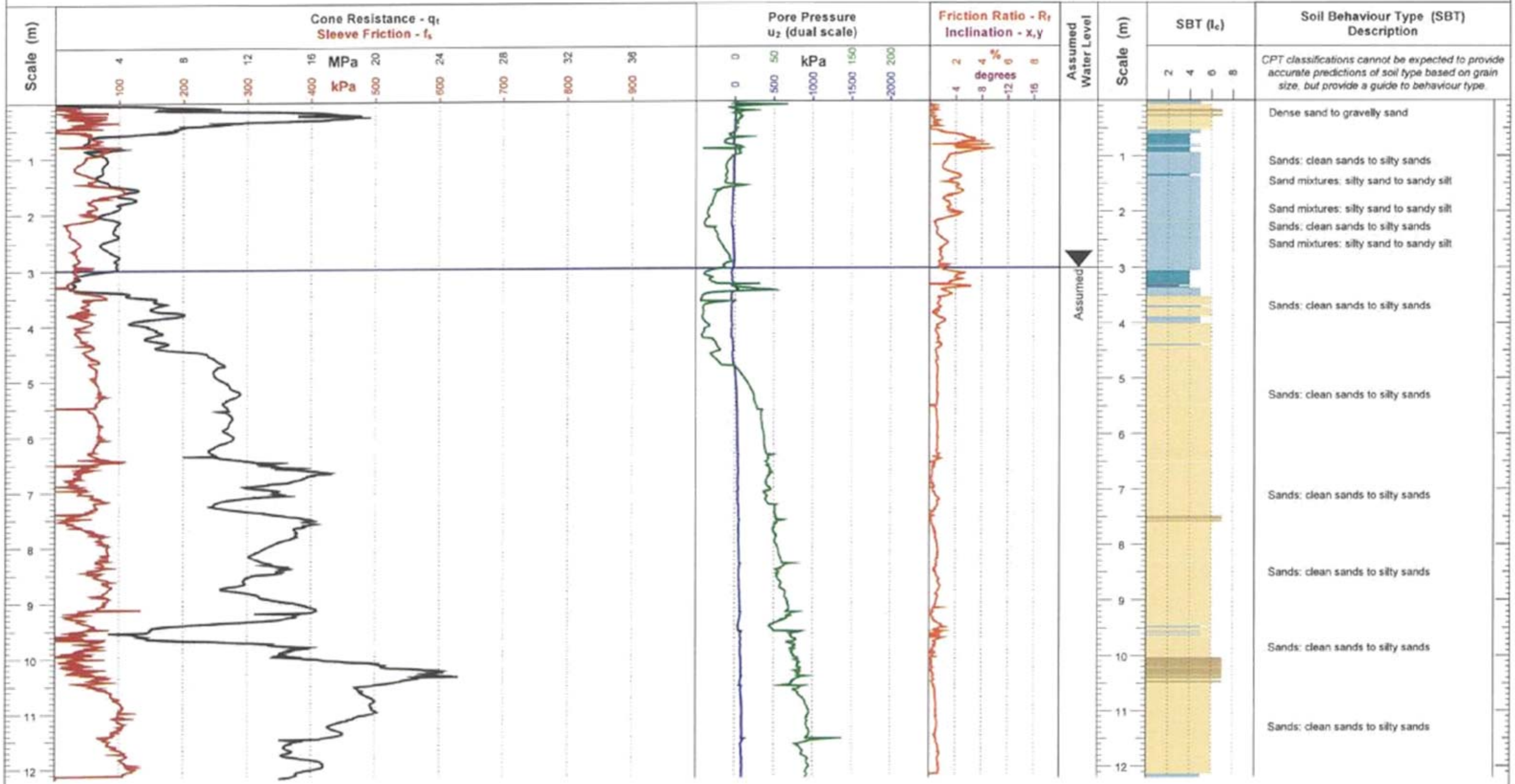


Client: Hiway Geotechnical	North (m): -	Remarks:	CPT Number: CPT-Y1250
Project Name: Willowlea Retirement Village	East (m): -		
Location: 27 Shirley Road, Shirley, Christchurch	Elevation (m): -		
Project Engineer: Saad Ahmed	Location Method:	Operator: JC / Cone Area Ratio: 0.85	Project ID: 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	Hole Depth (m): 12.20	Cone Number: S15CFIIP.835 / Sleeve Offset (m): 0	Date: 12/01/2016 / Page 1 of 1

CONE PENETRATION TEST (CPT) LOG

Hiway GeoTechnical

GROUND INVESTIGATION

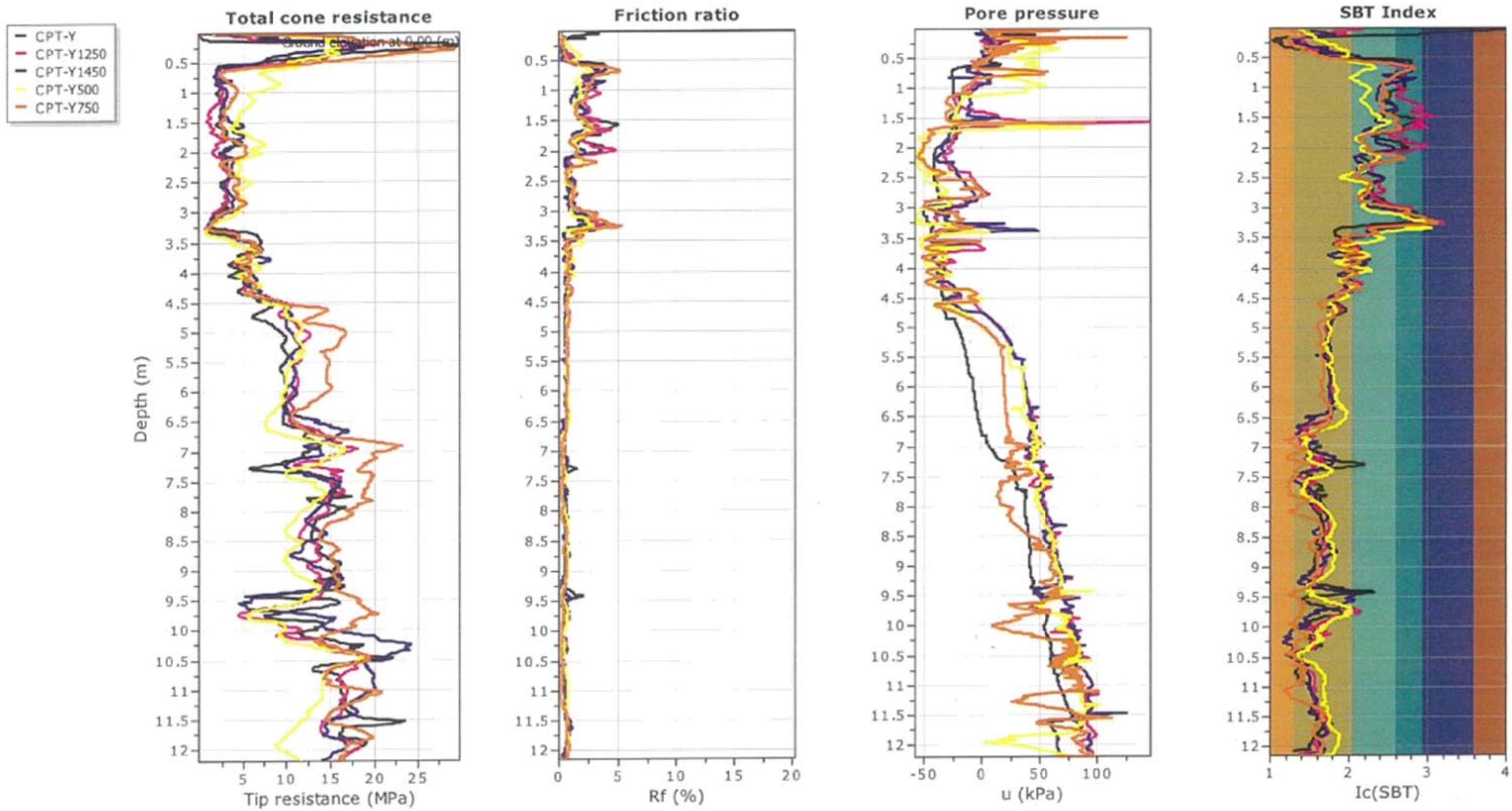


Client: Hiway Geotechnical	**North (m):** -	**Remarks:**	**CPT Number:** CPT-Y1450
Project Name: Willowlea Retirement Village	**East (m):** -		
Location: 27 Shirley Road, Shirley, Christchurch	**Elevation (m):** -		
Project Engineer: Saad Ahmed	**Location Method:**	**Operator:** JC / **Cone Area Ratio:** 0.85	**Project ID:** 15-169
Contractor: Ground Investigation Ltd. www.g-i.co.nz	**Hole Depth (m):** 12.20	**Cone Number:** S15CFIIP.835 / **Sleeve Offset (m):** 0	**Date:** 1/01/2003 / Page 1 of 1

Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

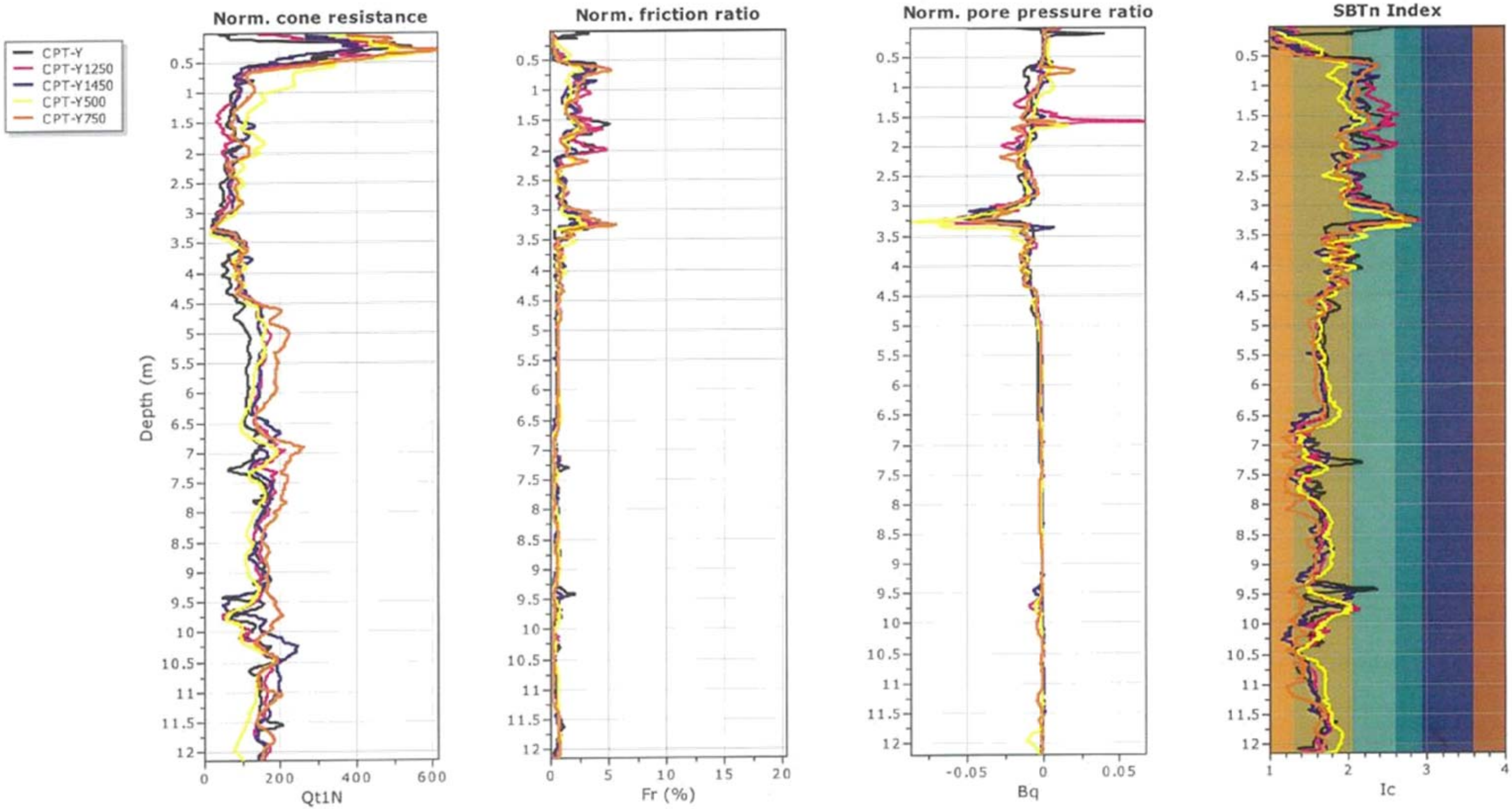
Overlay basic interpretation plots



Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

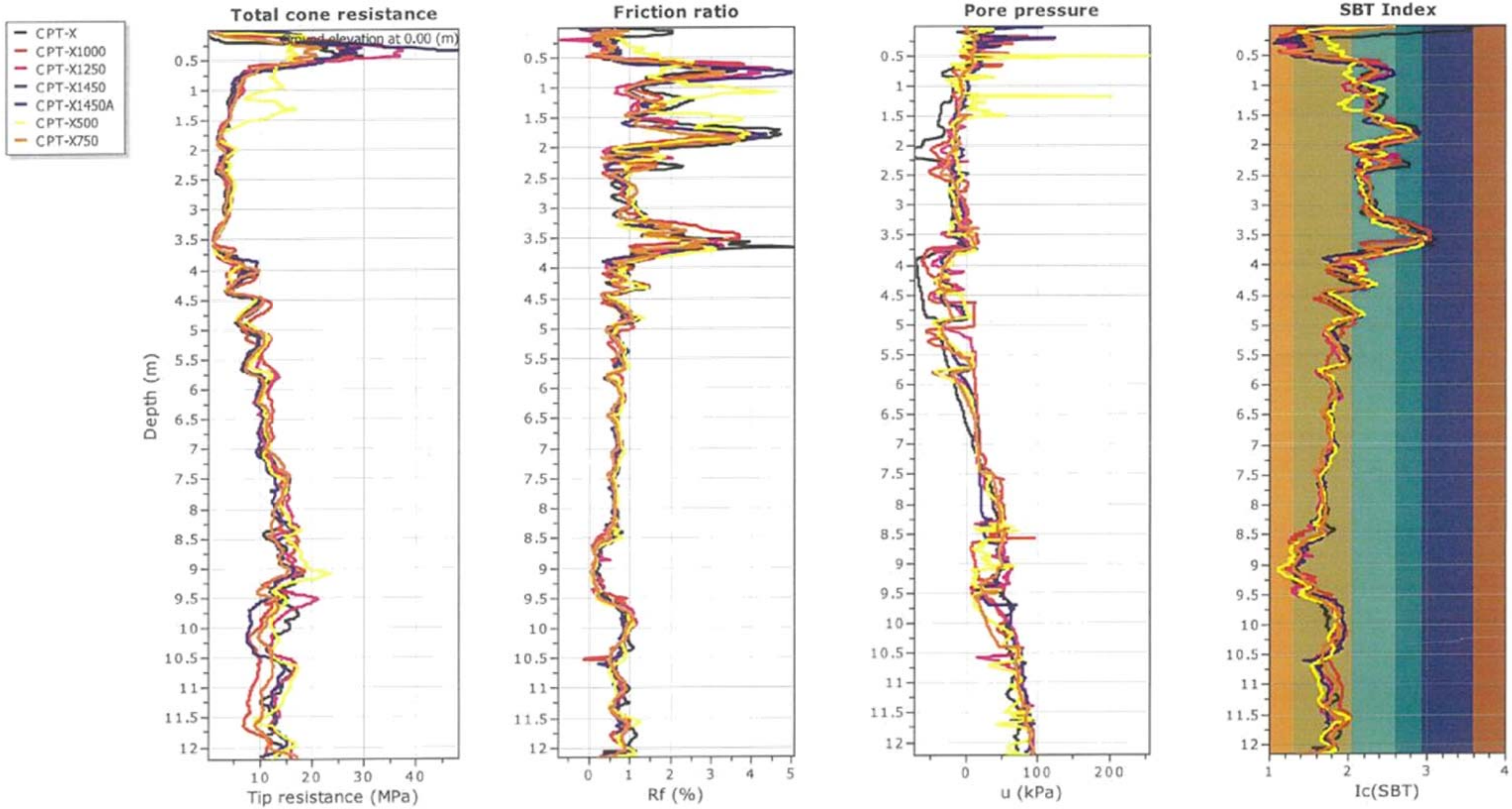
Normalized basic plots



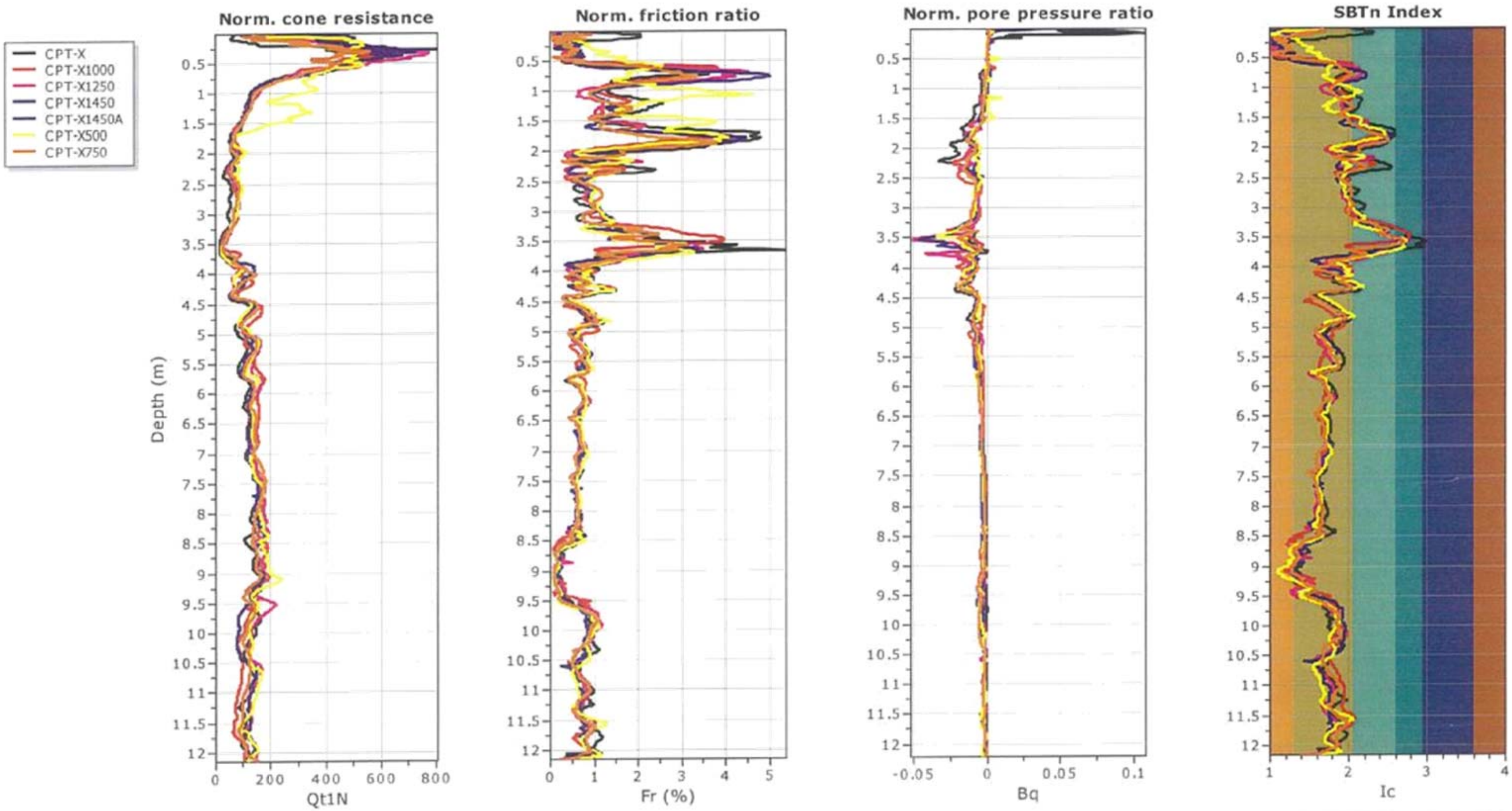
Ground Investigation
PO Box 104 089 Licoln North, Auckland, 0654
Ph: (09) 950 1919, (021) 191 6000
http://www.g-i.co.nz

Project: 15-169 - Willowlea Retirement Home
Location: 27 Shirley Road, Shirley, Christchurch

Overlay basic interpretation plots

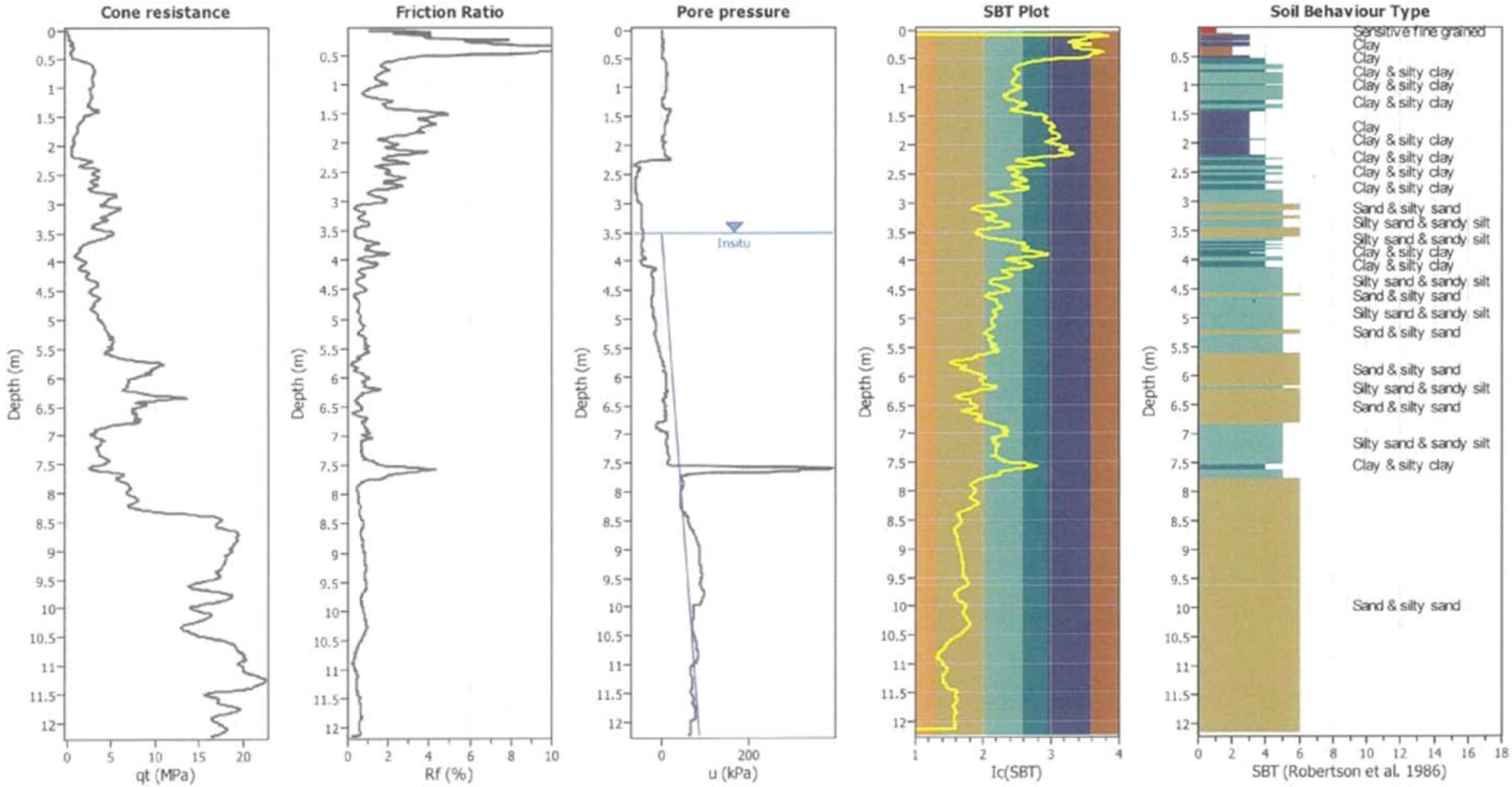


Normalized basic plots



Appendix J – Research Liquefaction Analysis (CPT)

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A		
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A		

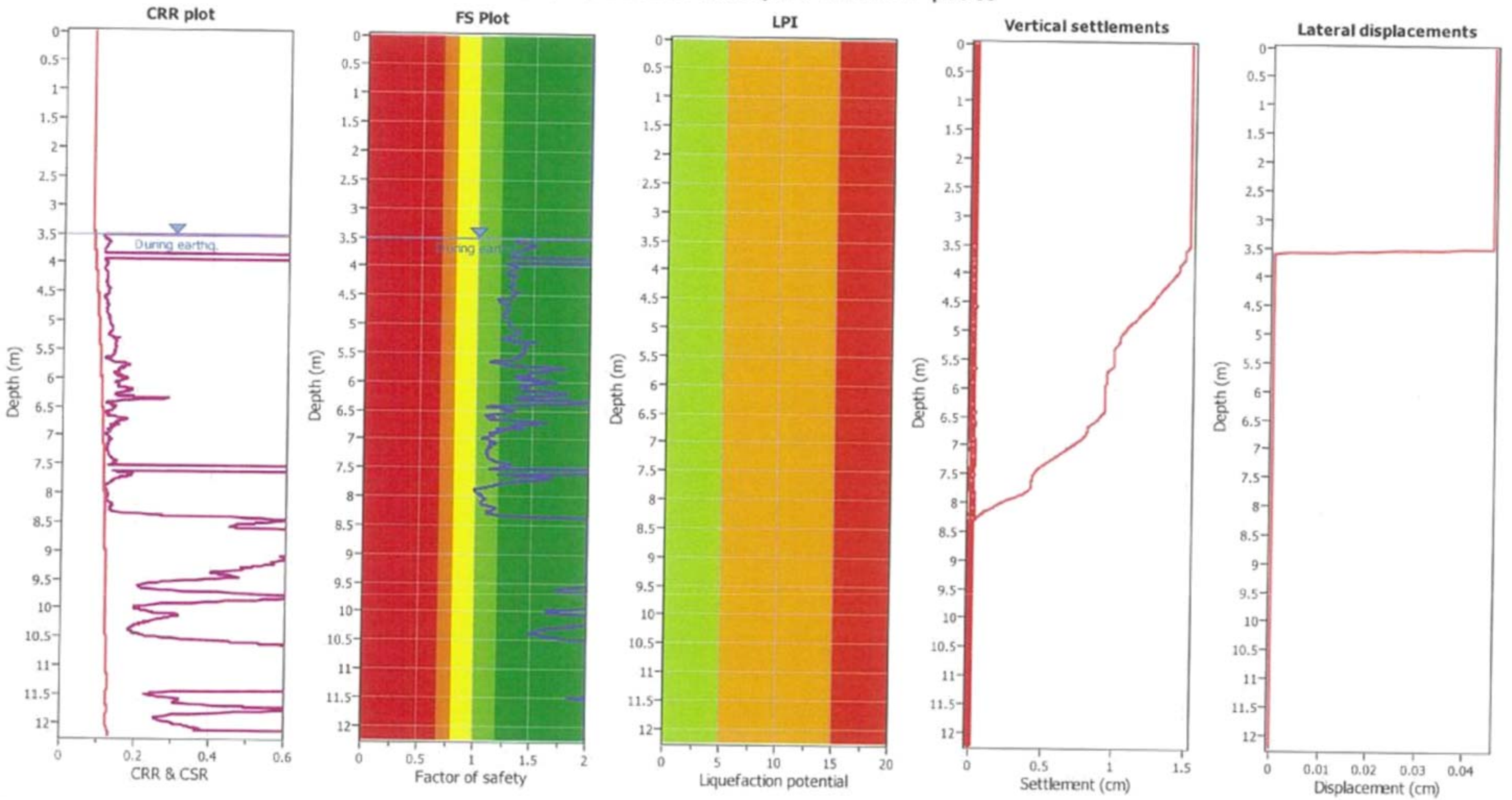
SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

This software is licensed to:

CPT name: CPT-01

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

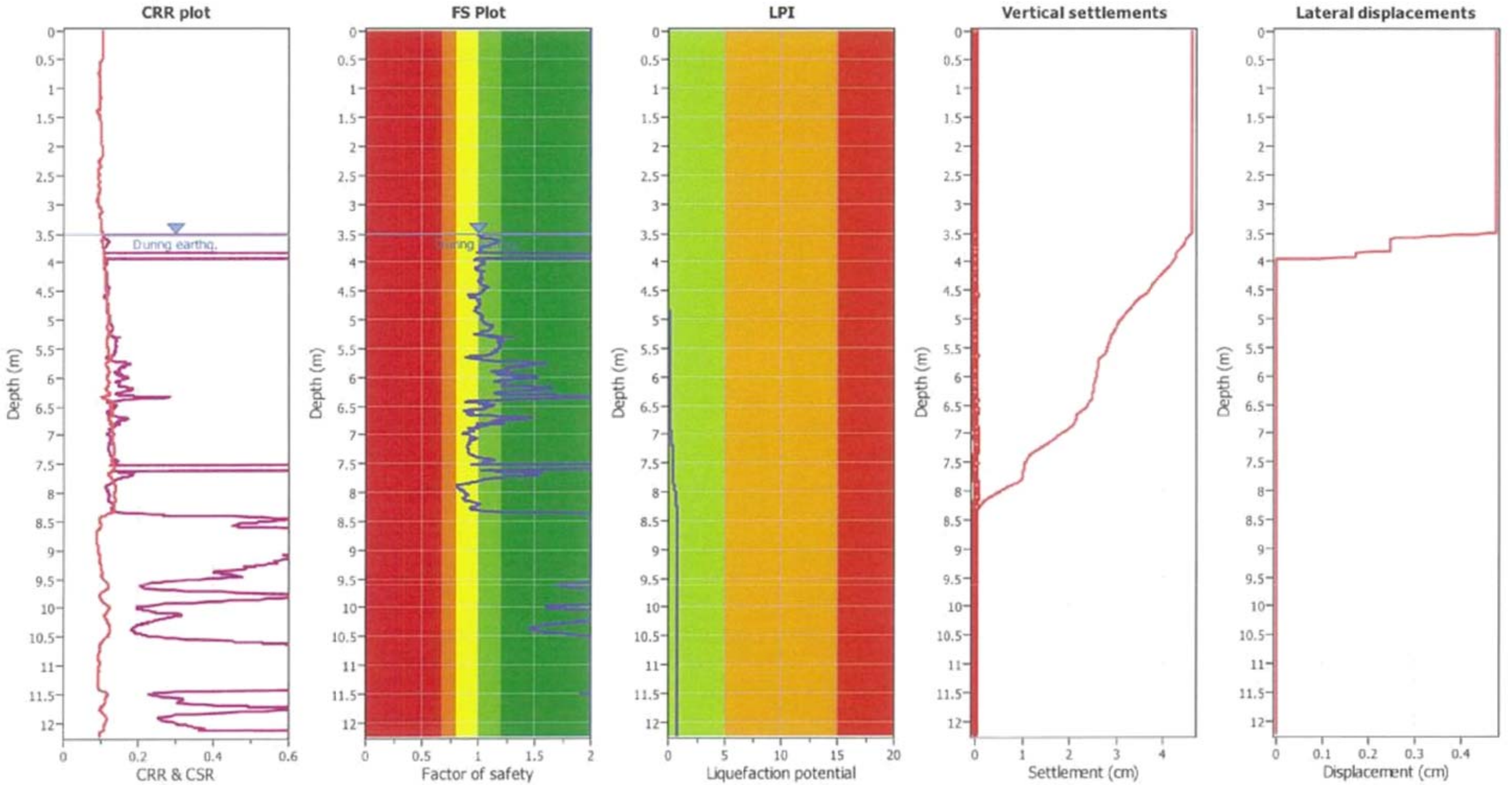
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

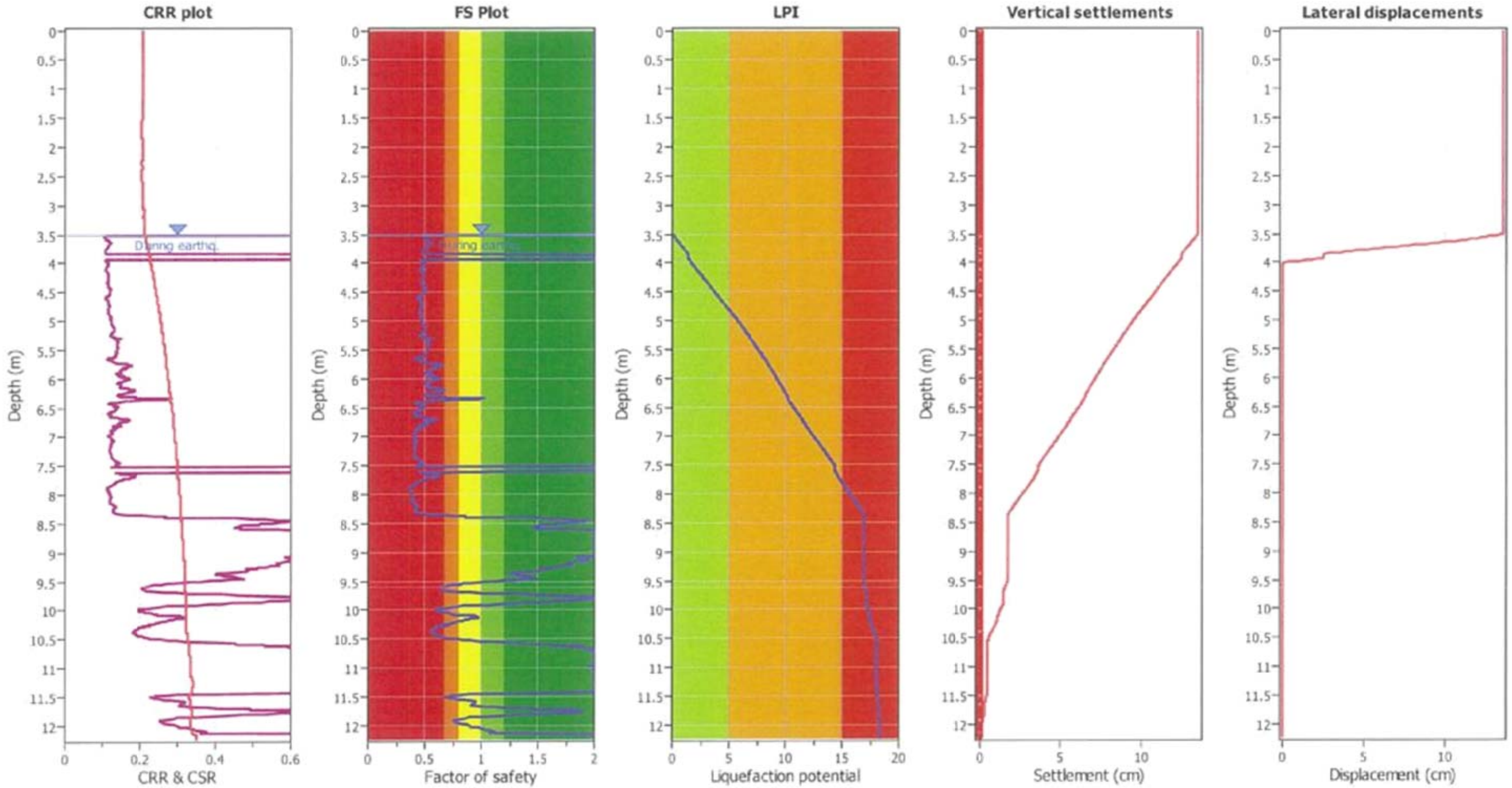
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
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- Liquefaction and no liq. are equally likely
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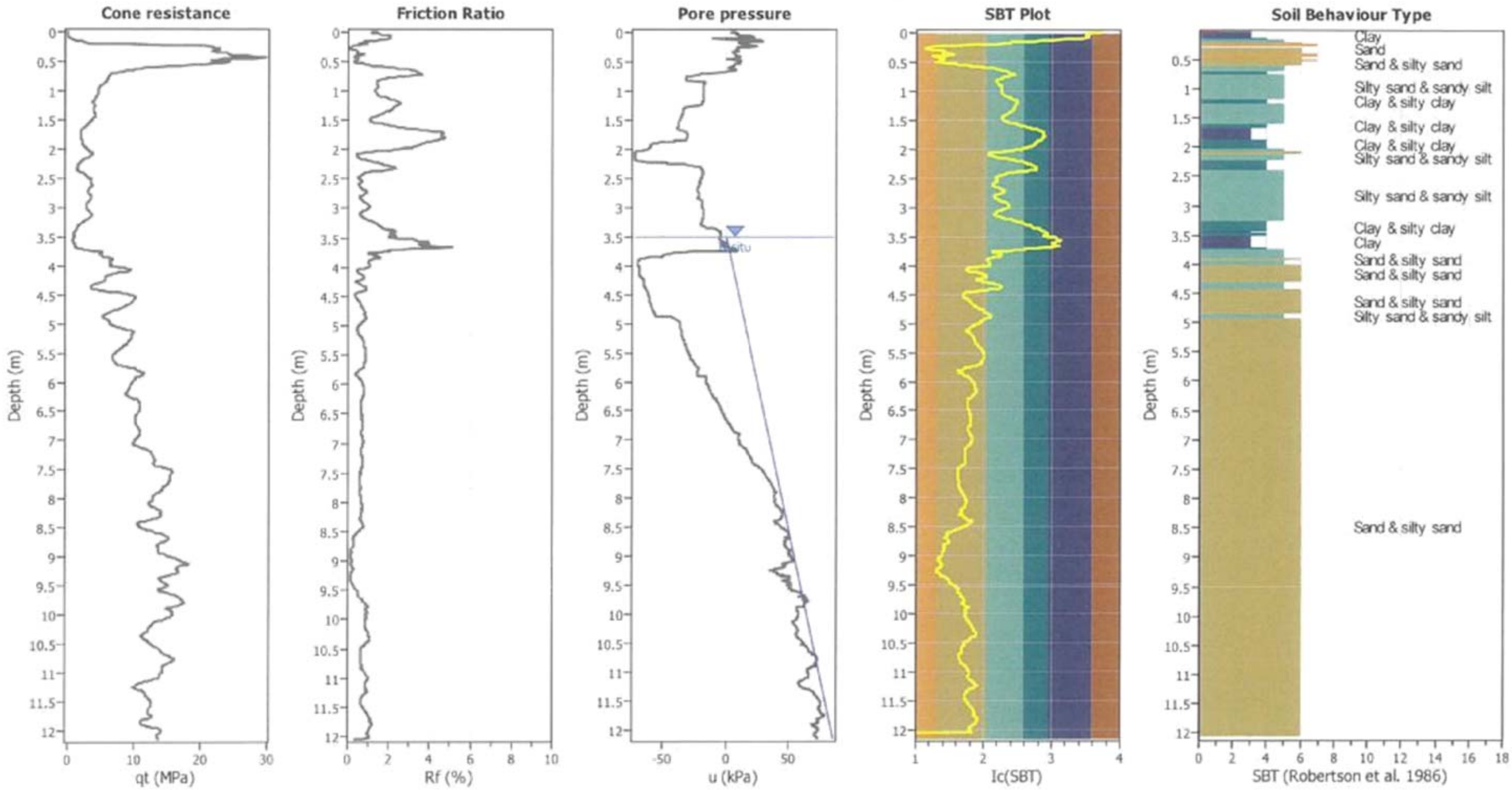
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plots



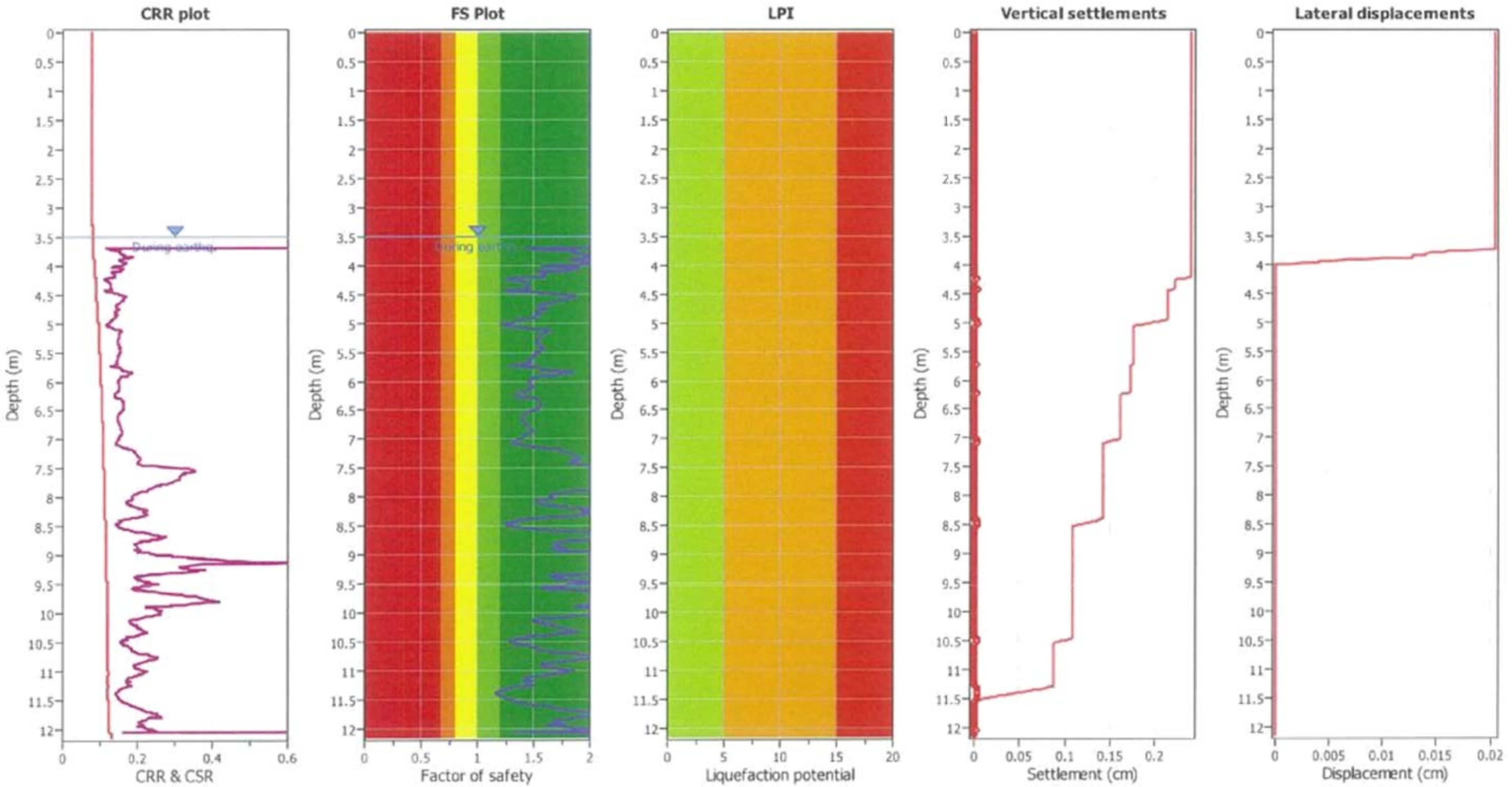
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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LPI color scheme

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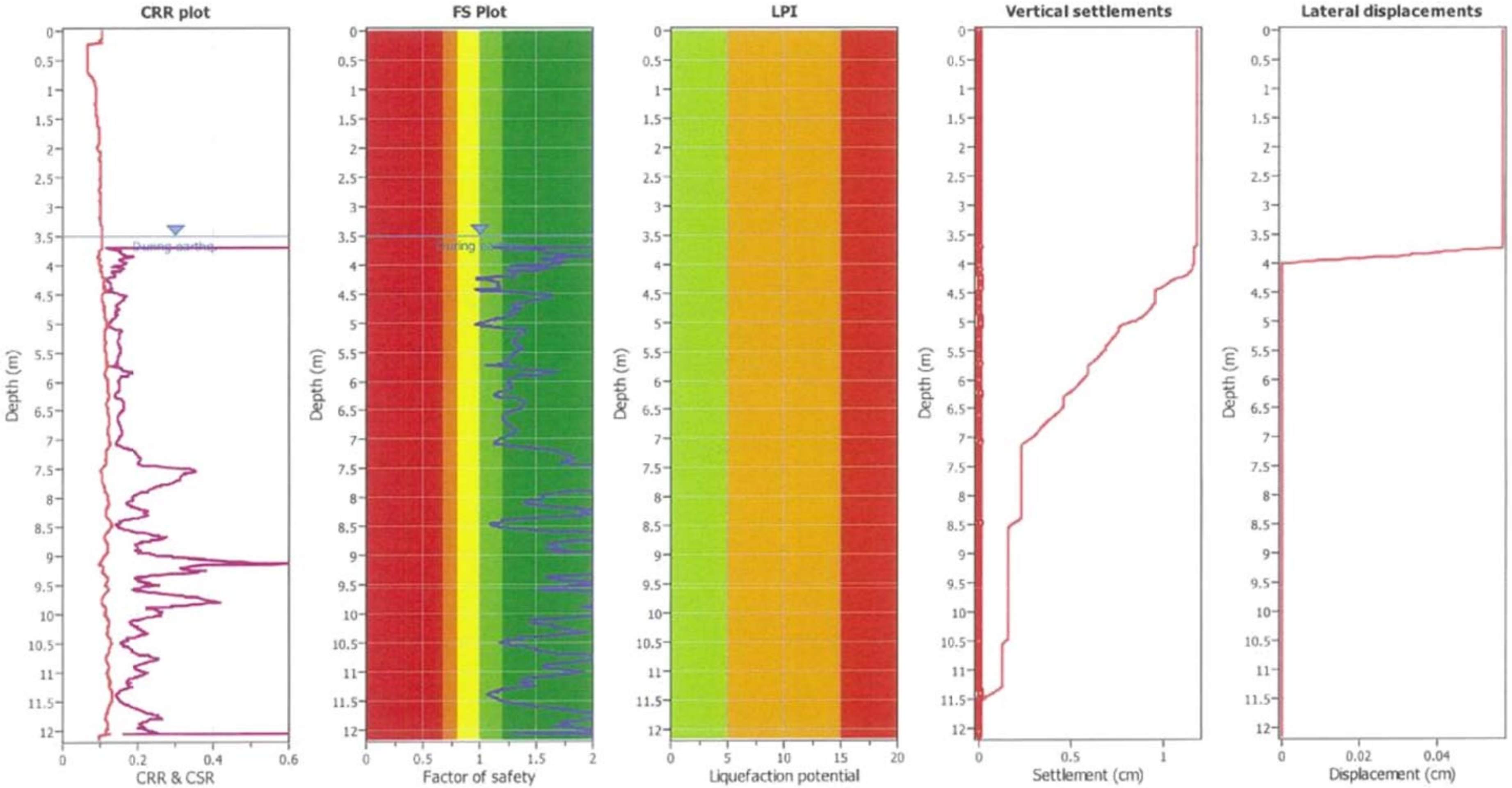
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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LPI color scheme

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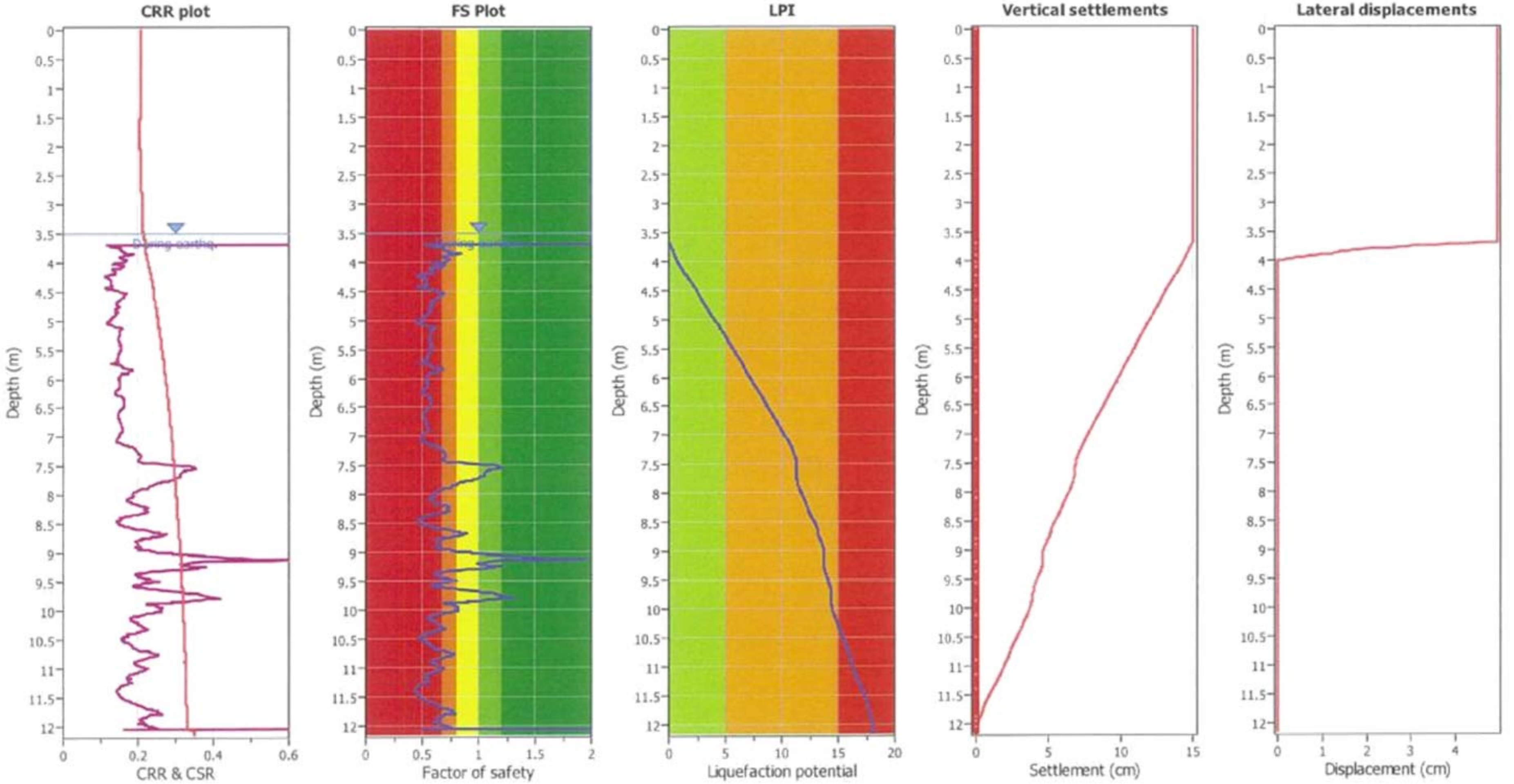
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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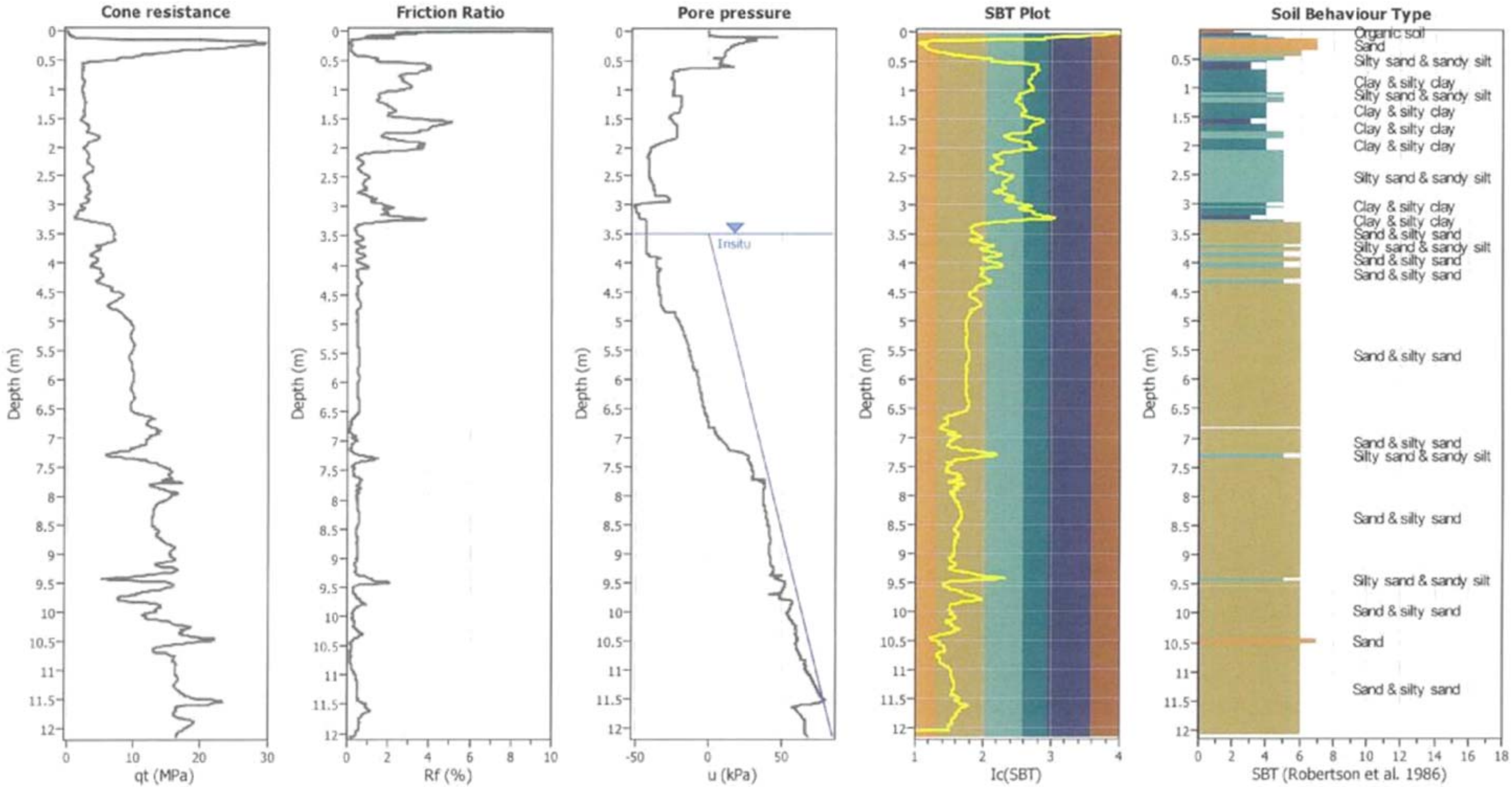
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plots



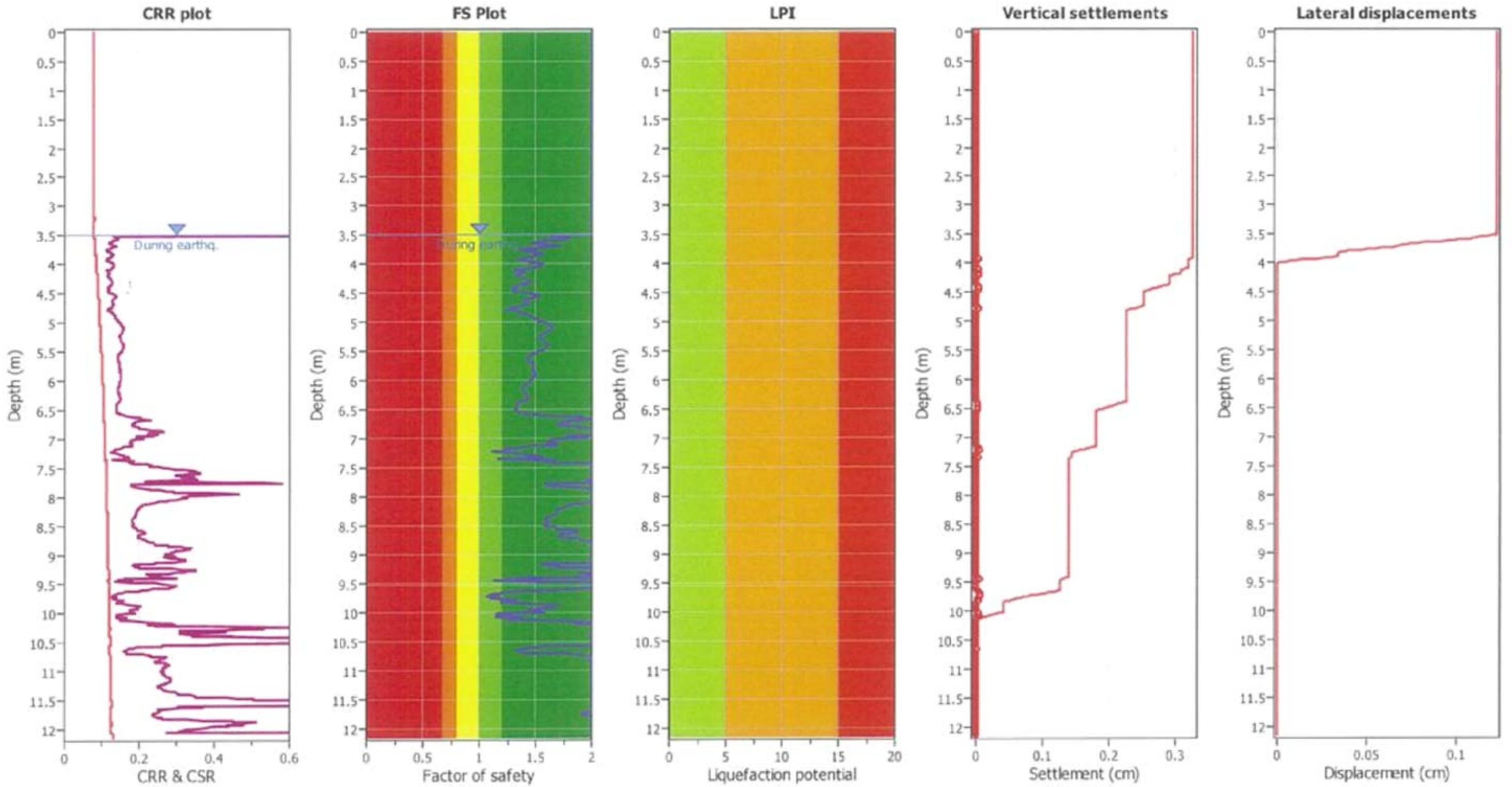
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)
Fines correction method:	B&I (2014)
Points to test:	Based on Ic value
Earthquake magnitude M_w:	7.50
Peak ground acceleration:	0.13
Depth to water table (insitu):	3.50 m

Depth to GWT (erthq.):	3.50 m
Average results interval:	3
Ic cut-off value:	2.60
Unit weight calculation:	Based on SBT
Use fill:	No
Fill height:	N/A

Fill weight:	N/A
Transition detect. applied:	No
K_σ applied:	Yes
Clay like behavior applied:	Sands only
Limit depth applied:	No
Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

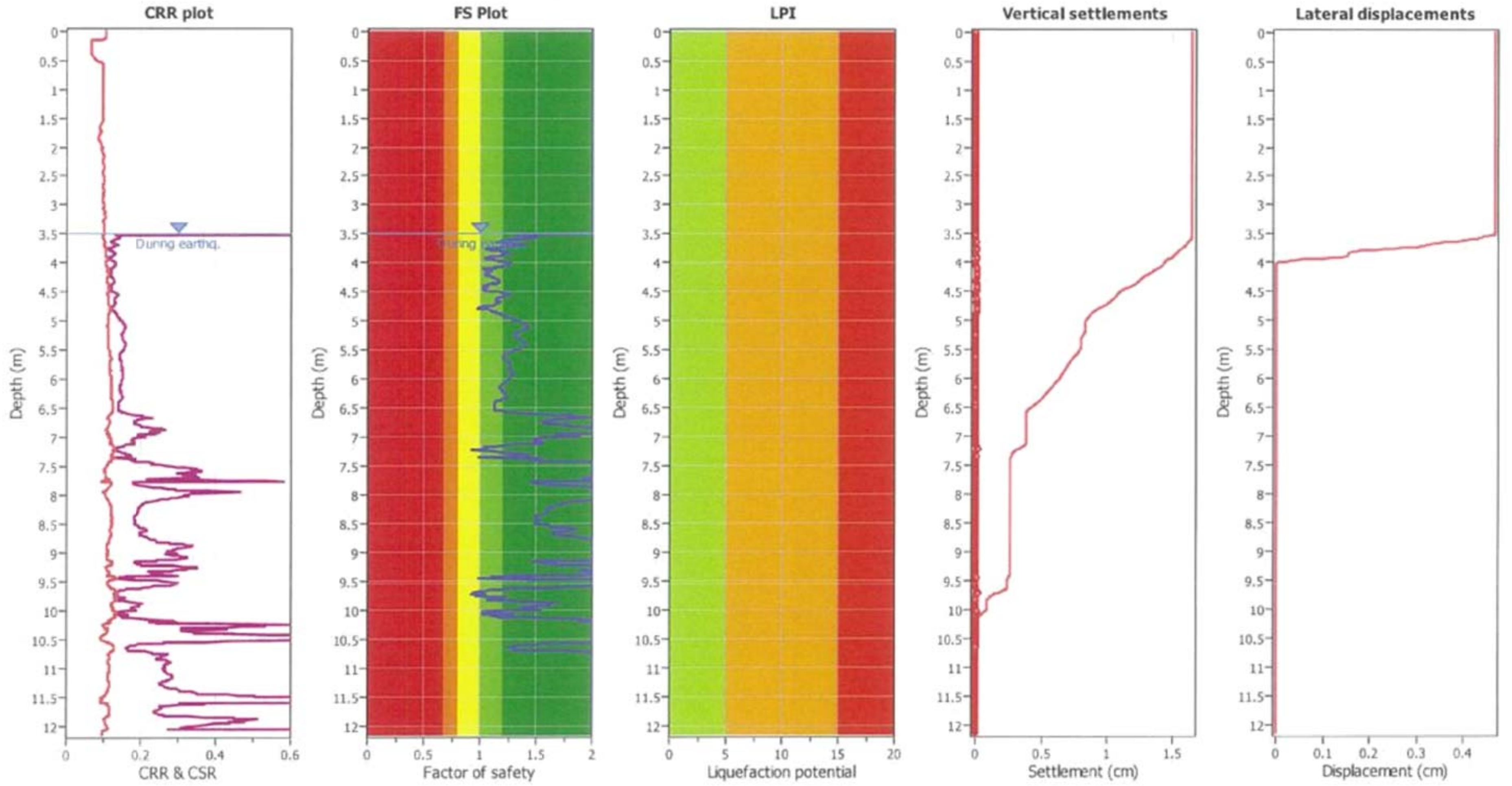
LPI color scheme

- Very high risk
- High risk
- Low risk

This software is licensed to:

CPT name: CPT-03

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

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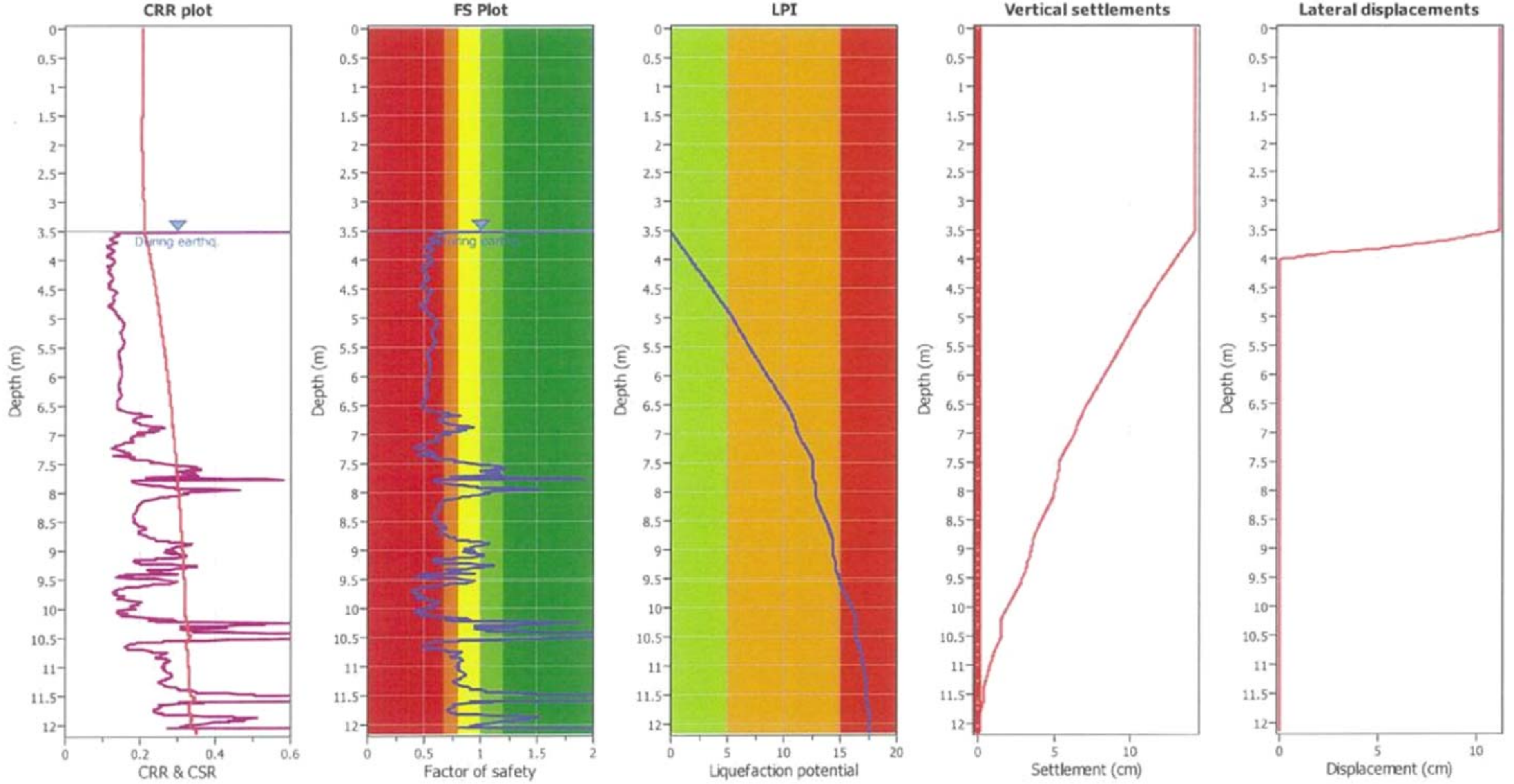
CLiq v.2.0.6.97 - CPT Liquefaction Assessment Software - Report created on: 01-Nov-16, 6:29:41 PM

Project file: I:\Documents\HFoy Documents\Willowlea post construction Testing\CPT X.Y.Z Pre DSM.clq

1

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

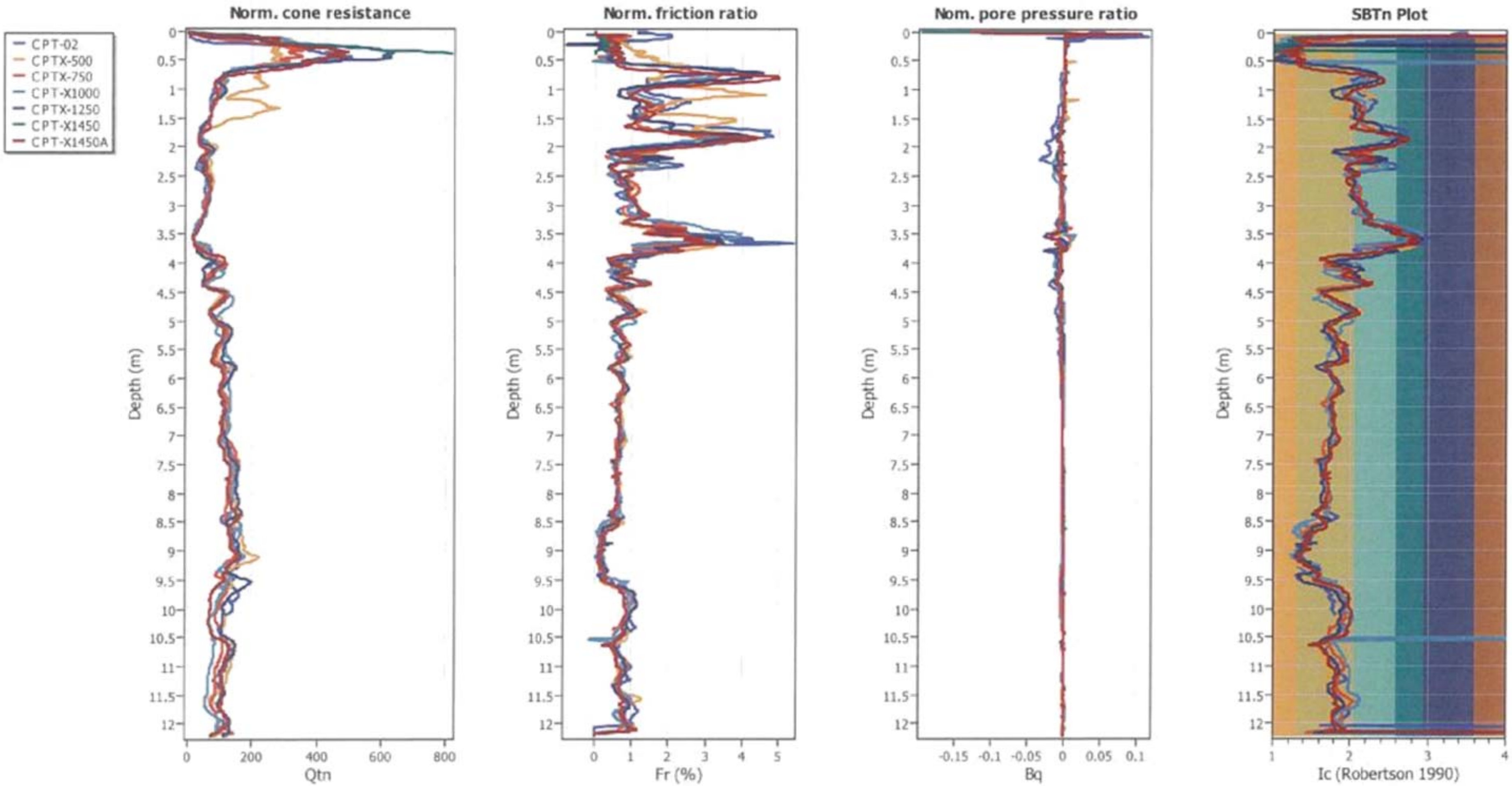
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Project: 27 Shirley Road, Shirley

Overlay Normalized Plots



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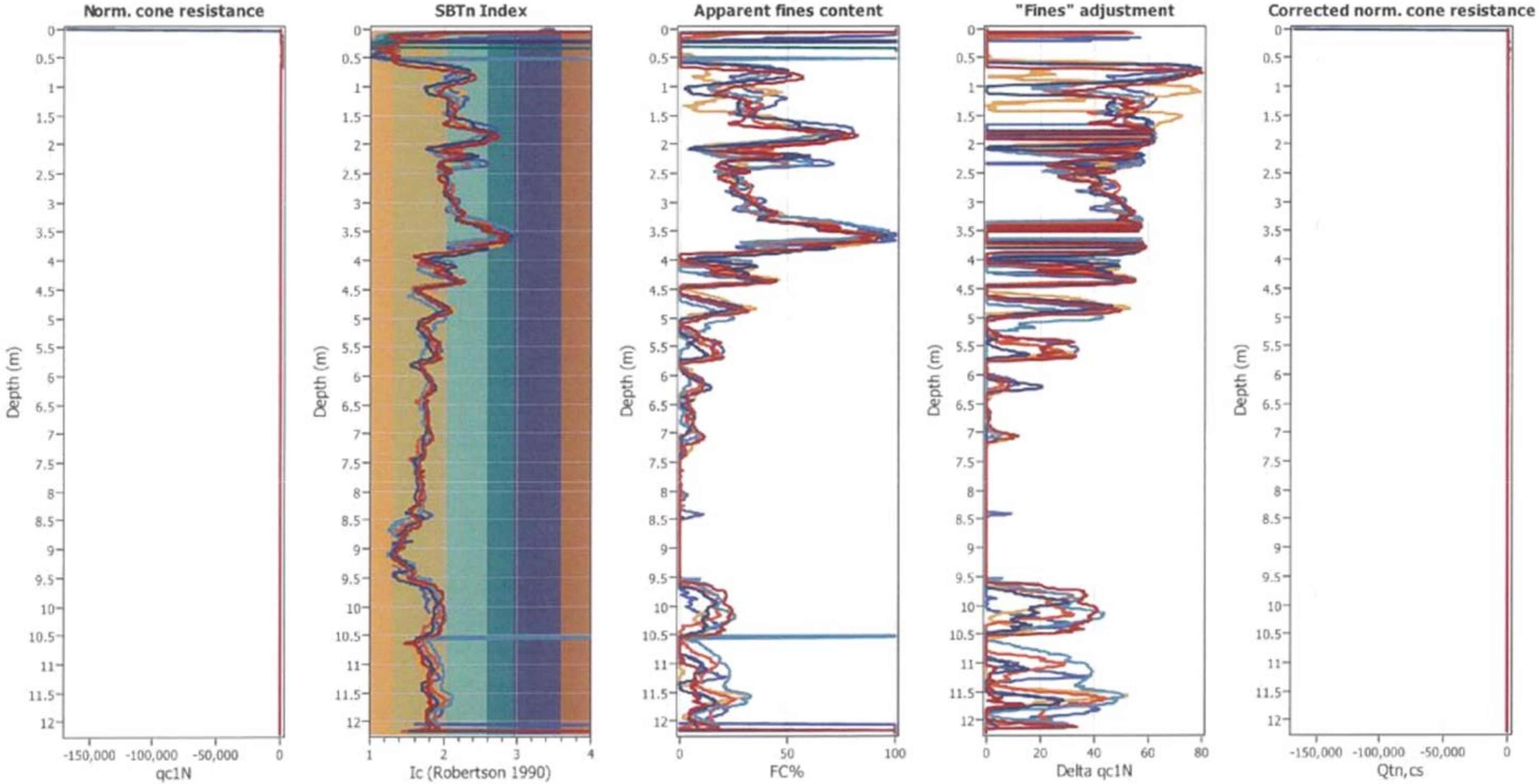
Project: 27 Shirley Road, Shirley

Overlay Intermediate Results



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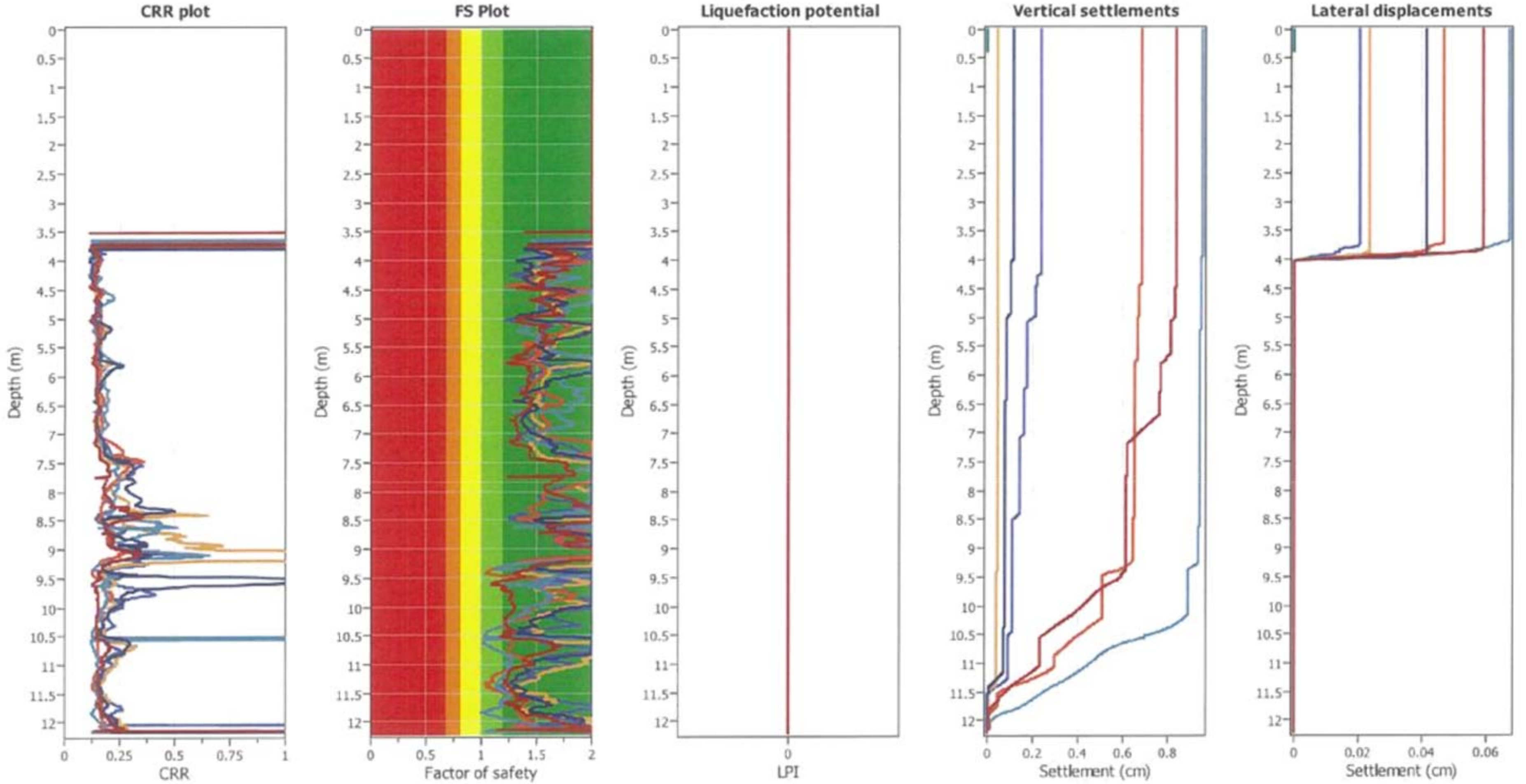
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

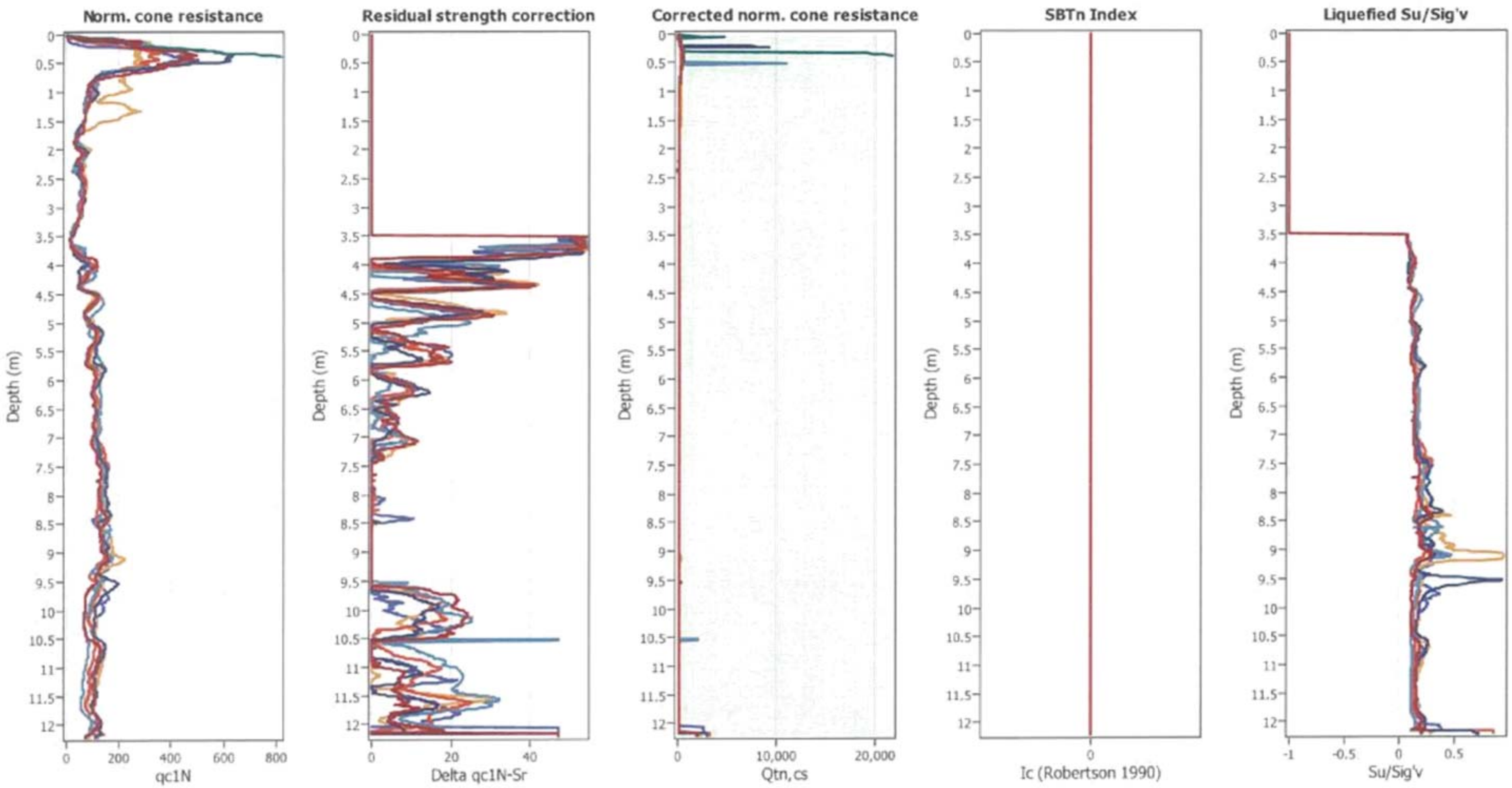
Project: 27 Shirley Road, Shirley

Overlay Cyclic Liquefaction Plots



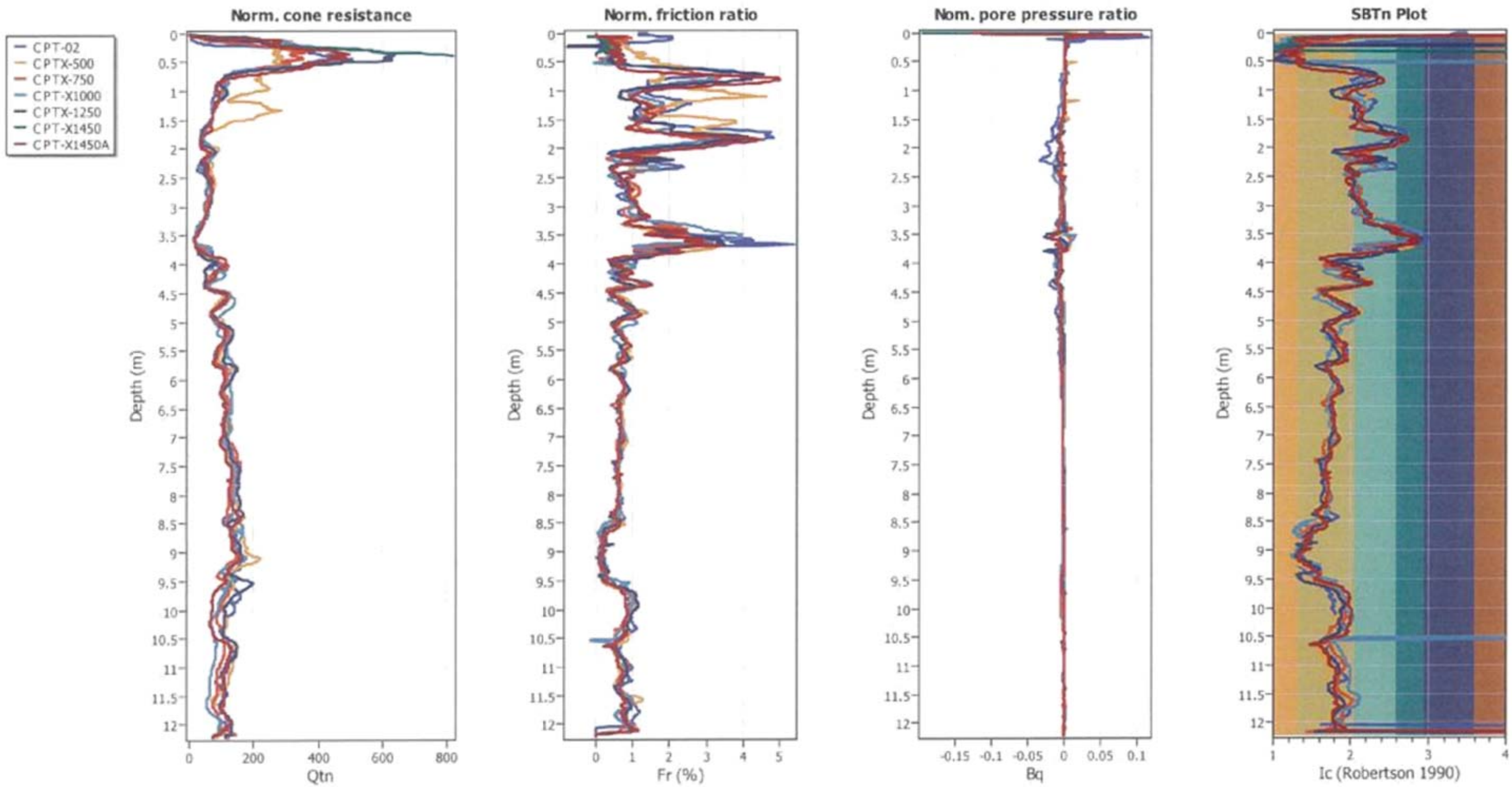
Project: 27 Shirley Road, Shirley

Overlay Strength Loss Plots



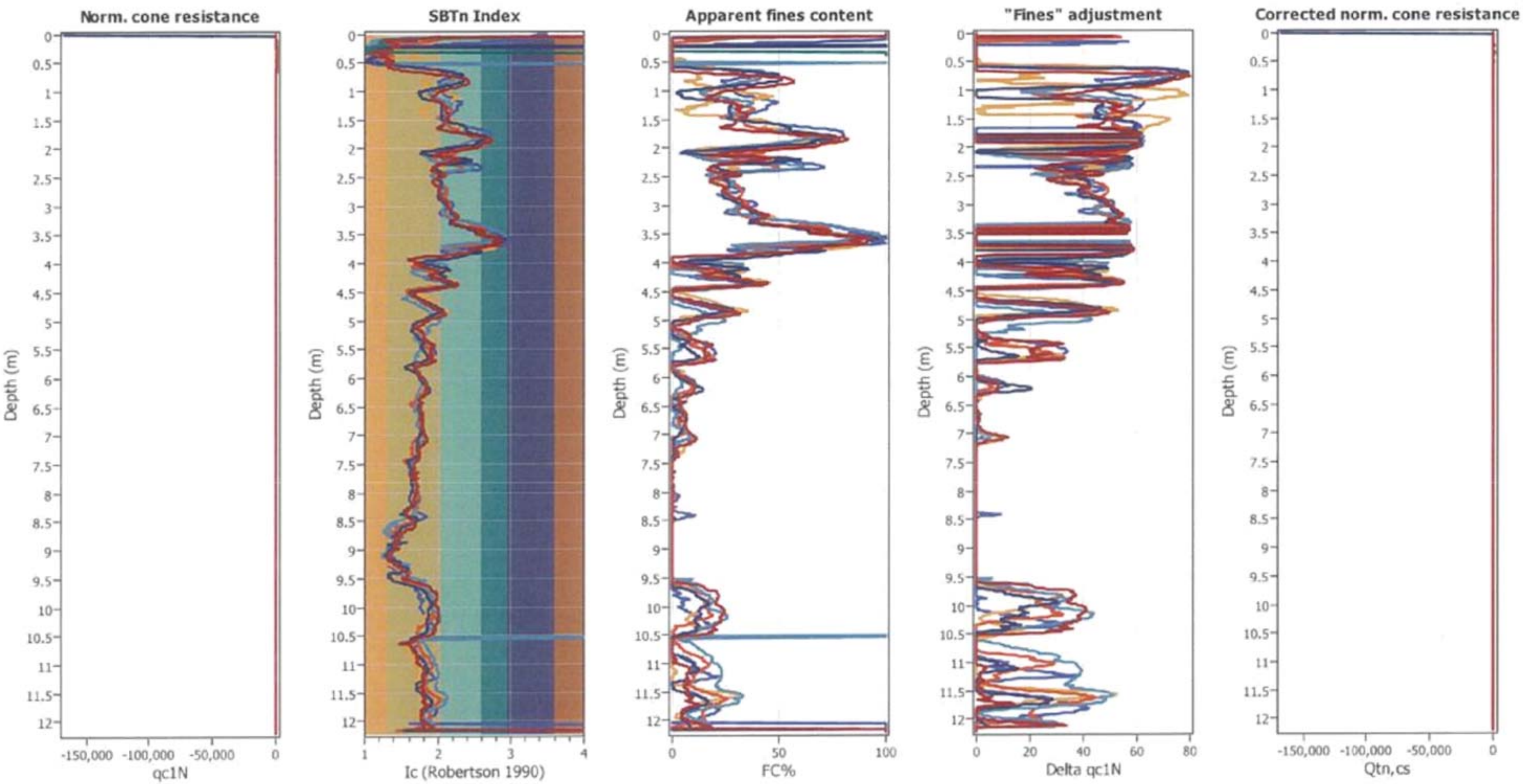
Project: 27 Shirley Road, Shirley

Overlay Normalized Plots



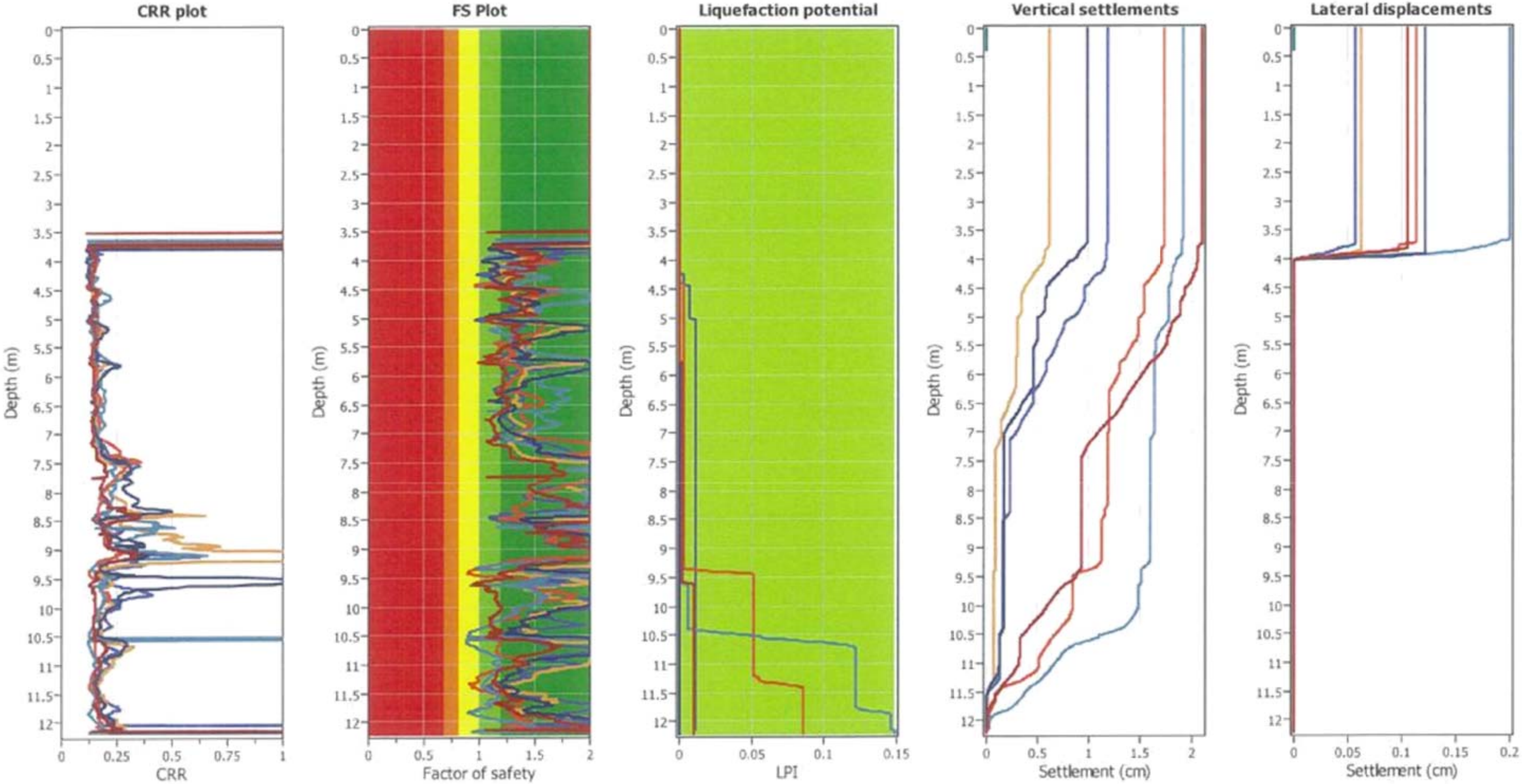
Project: 27 Shirley Road, Shirley

Overlay Intermediate Results



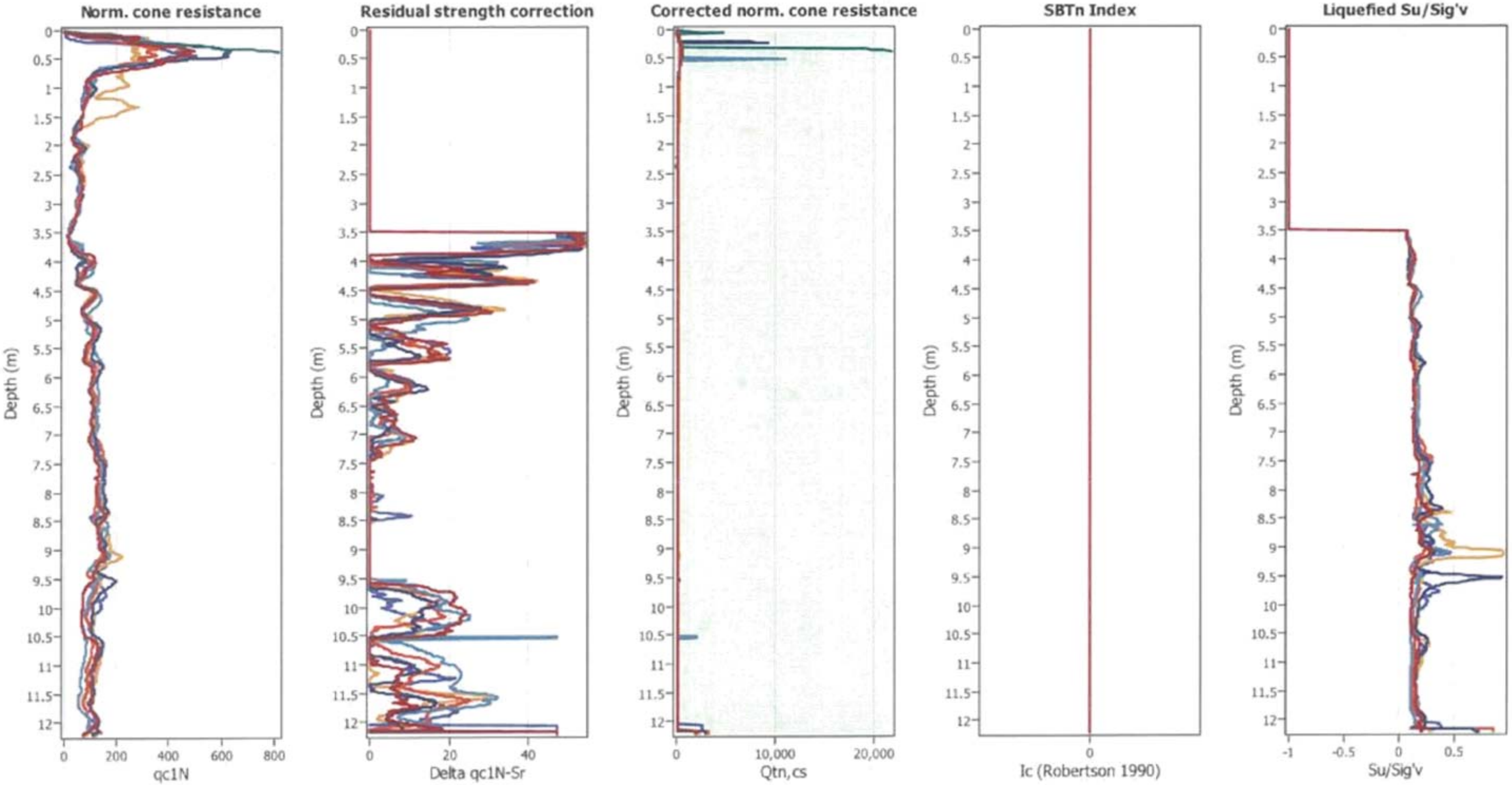
Project: 27 Shirley Road, Shirley

Overlay Cyclic Liquefaction Plots



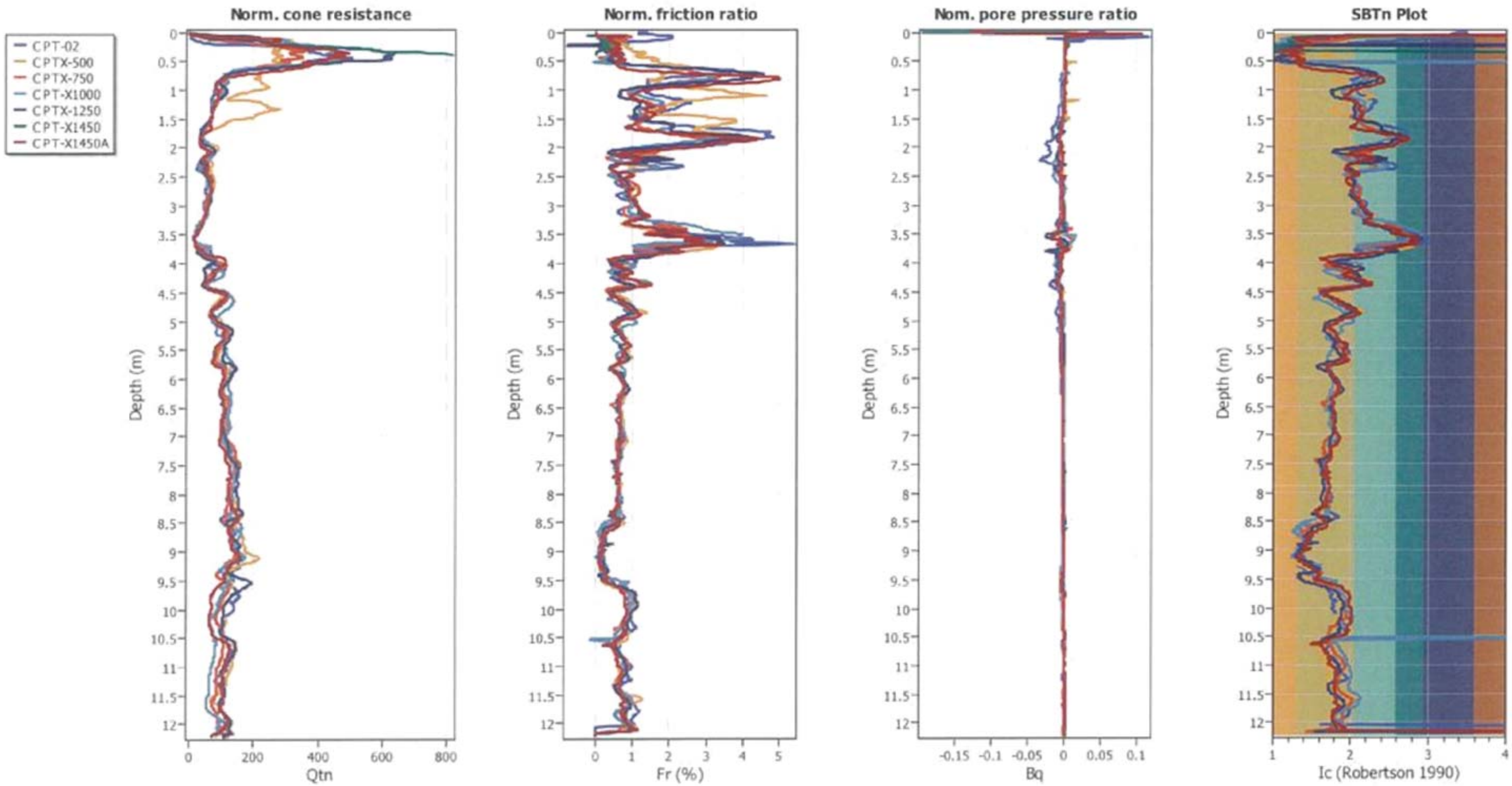
Project: 27 Shirley Road, Shirley

Overlay Strength Loss Plots



Project: 27 Shirley Road, Shirley

Overlay Normalized Plots



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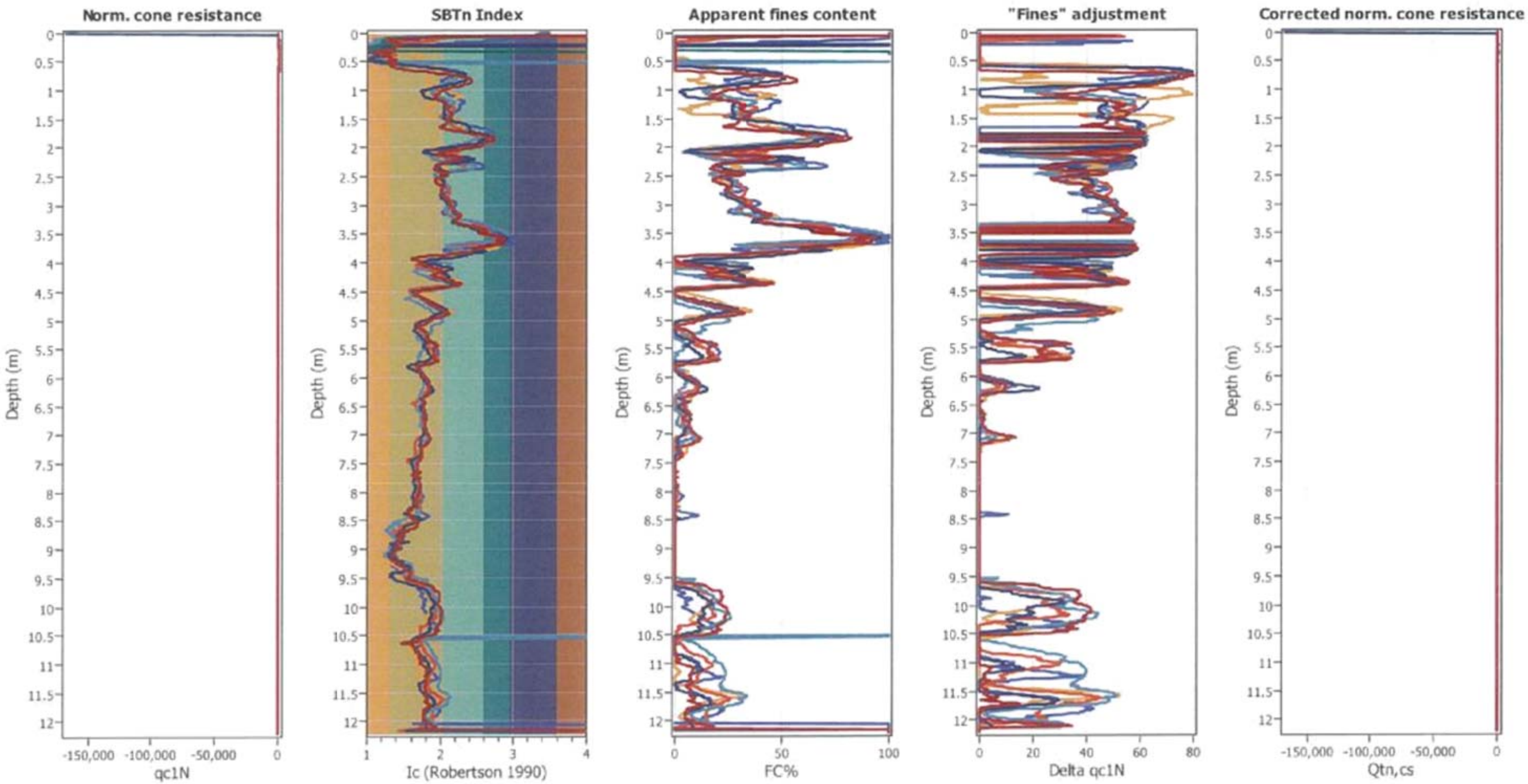
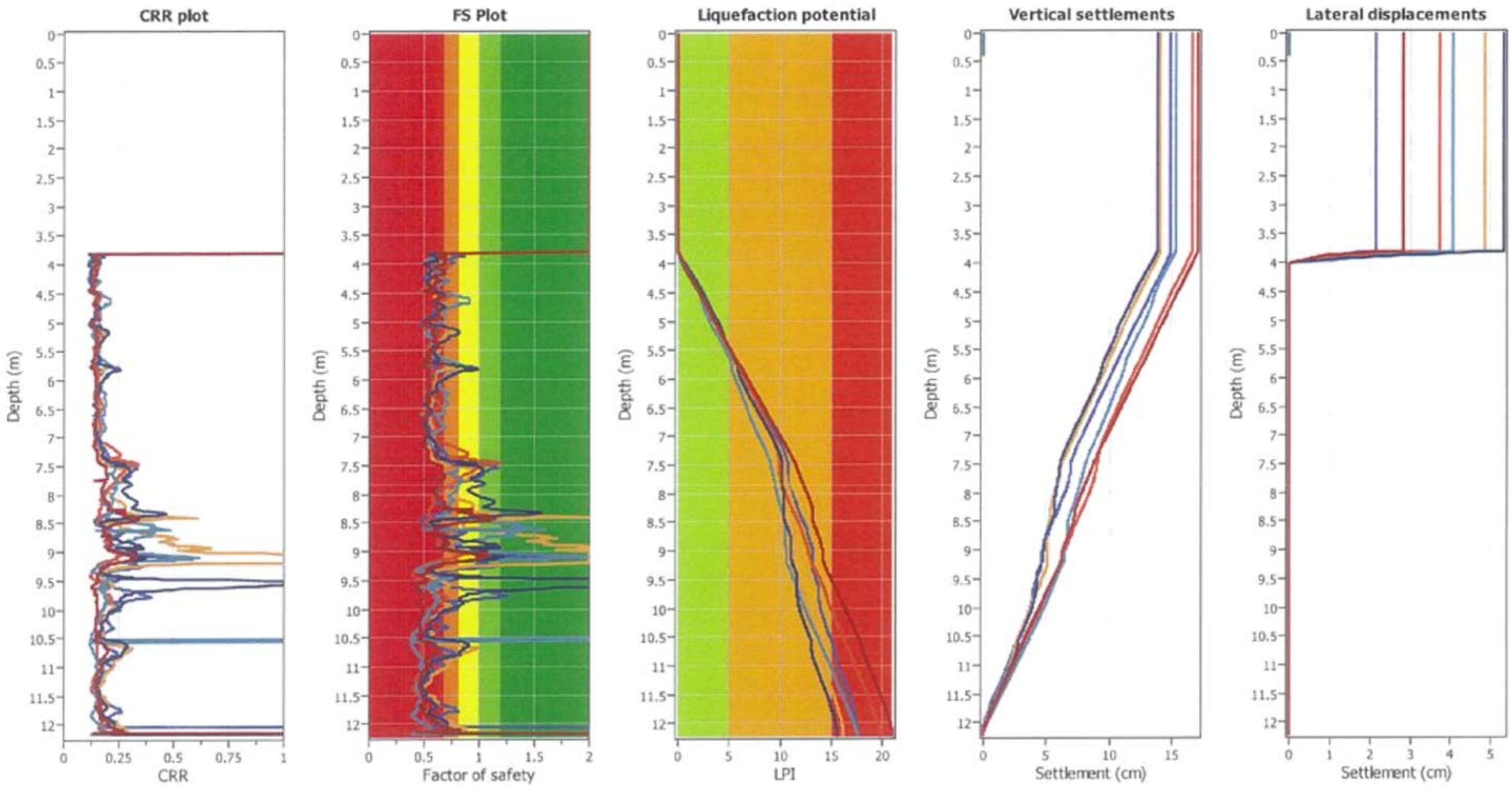
Project: 27 Shirley Road, Shirley

Overlay Intermediate Results



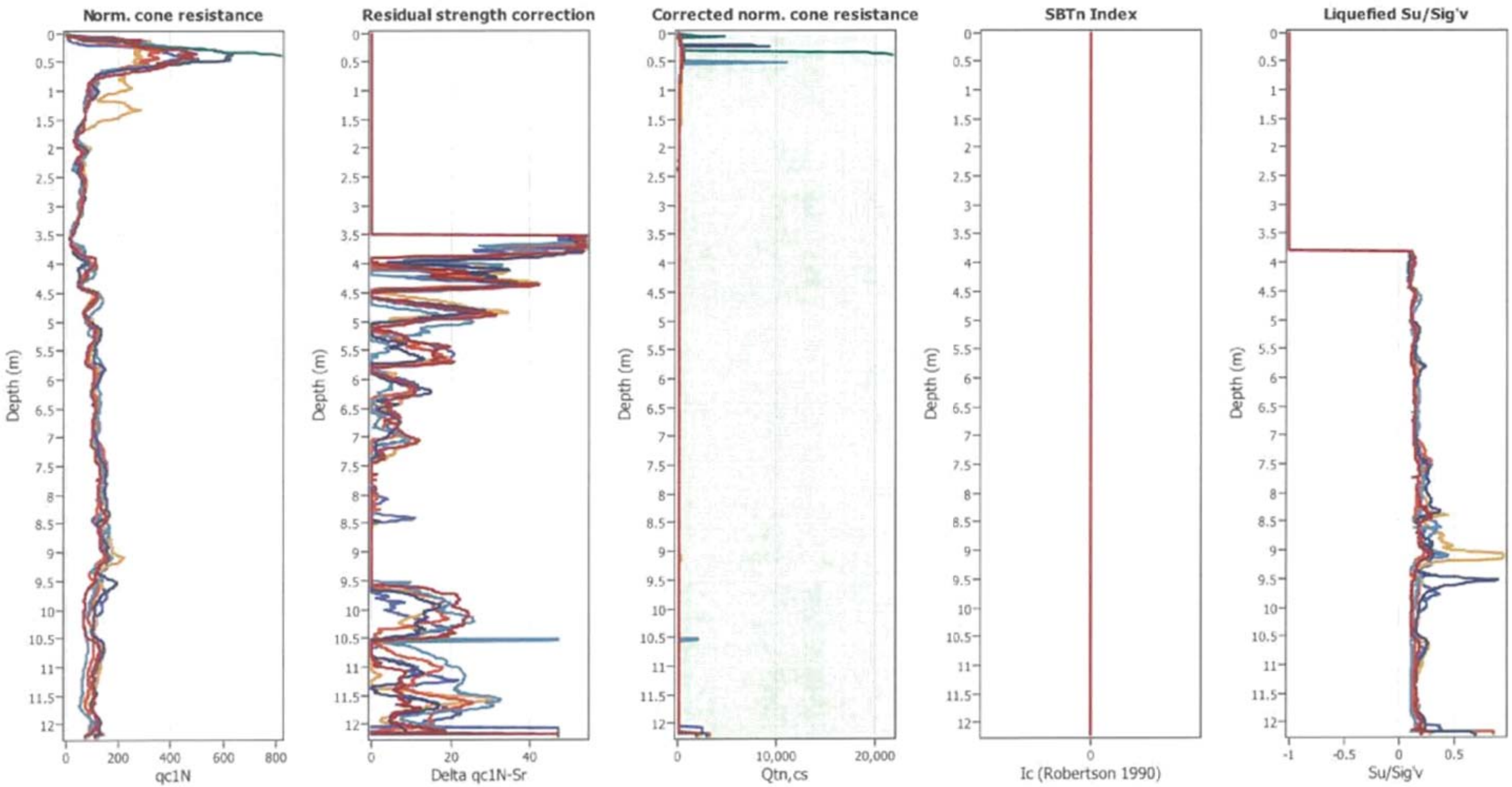
Project: 27 Shirley Road, Shirley

Overlay Cyclic Liquefaction Plots



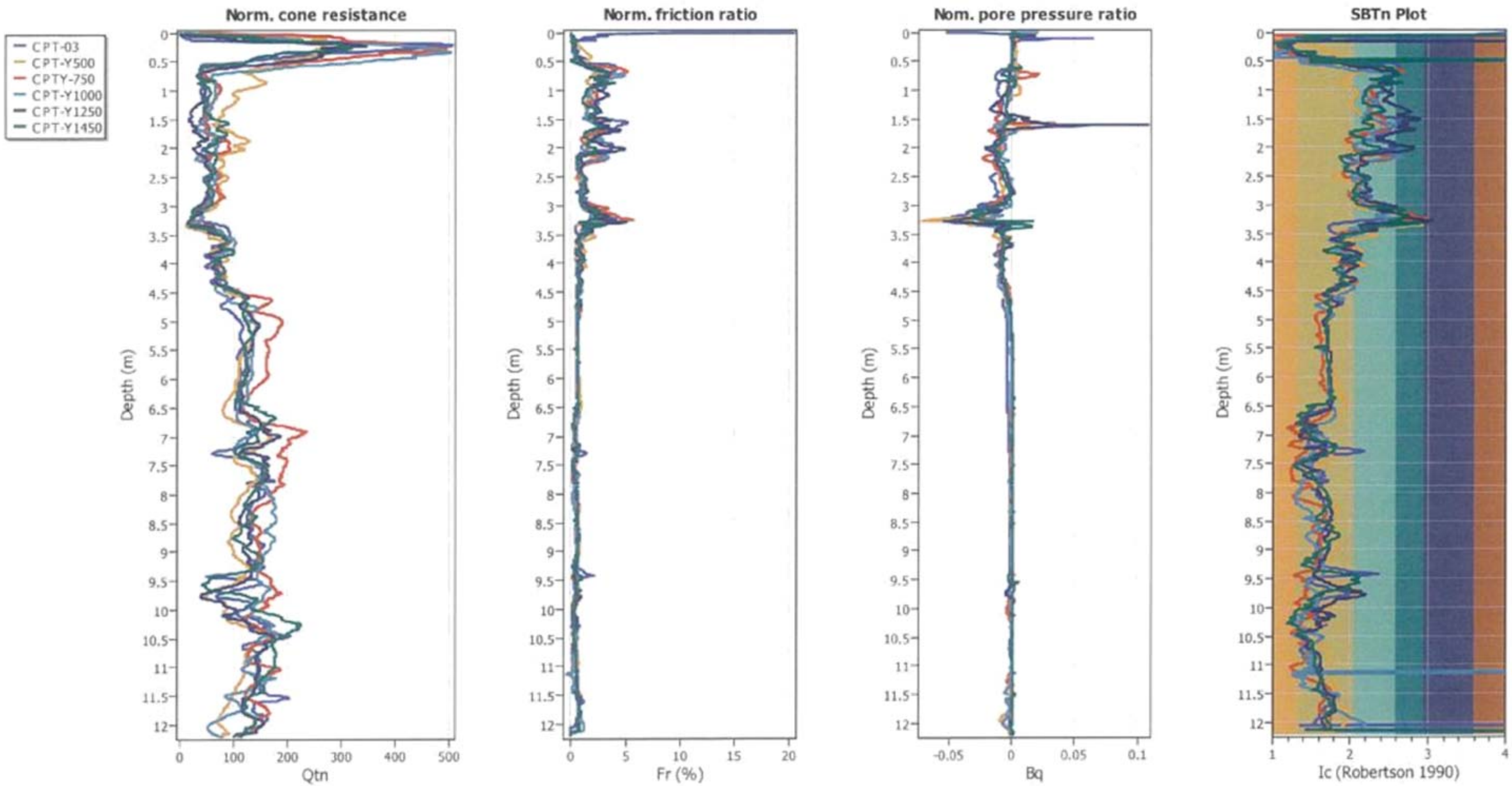
Project: 27 Shirley Road, Shirley

Overlay Strength Loss Plots



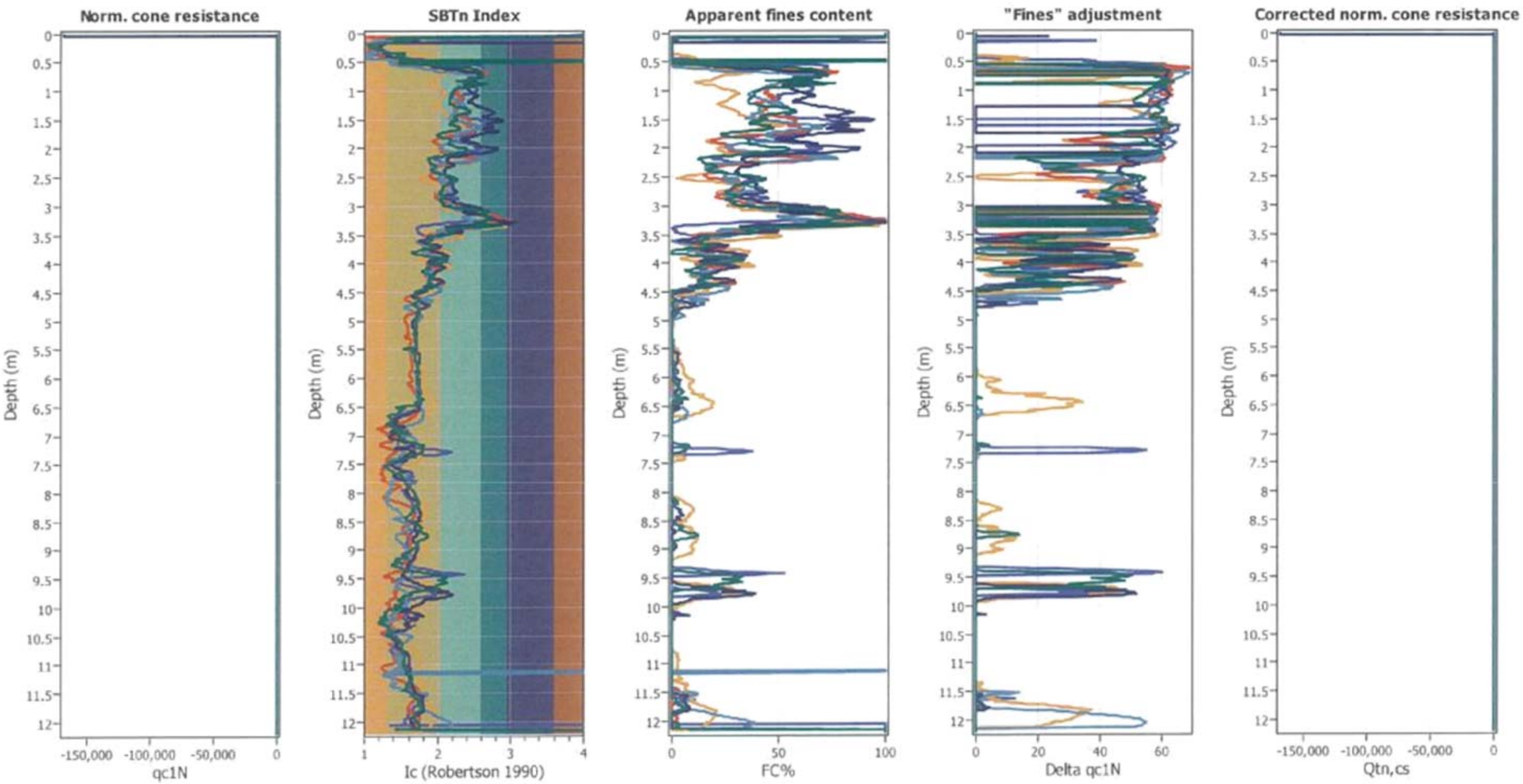
Project: 27 Shirley Road, Shirley

Overlay Normalized Plots



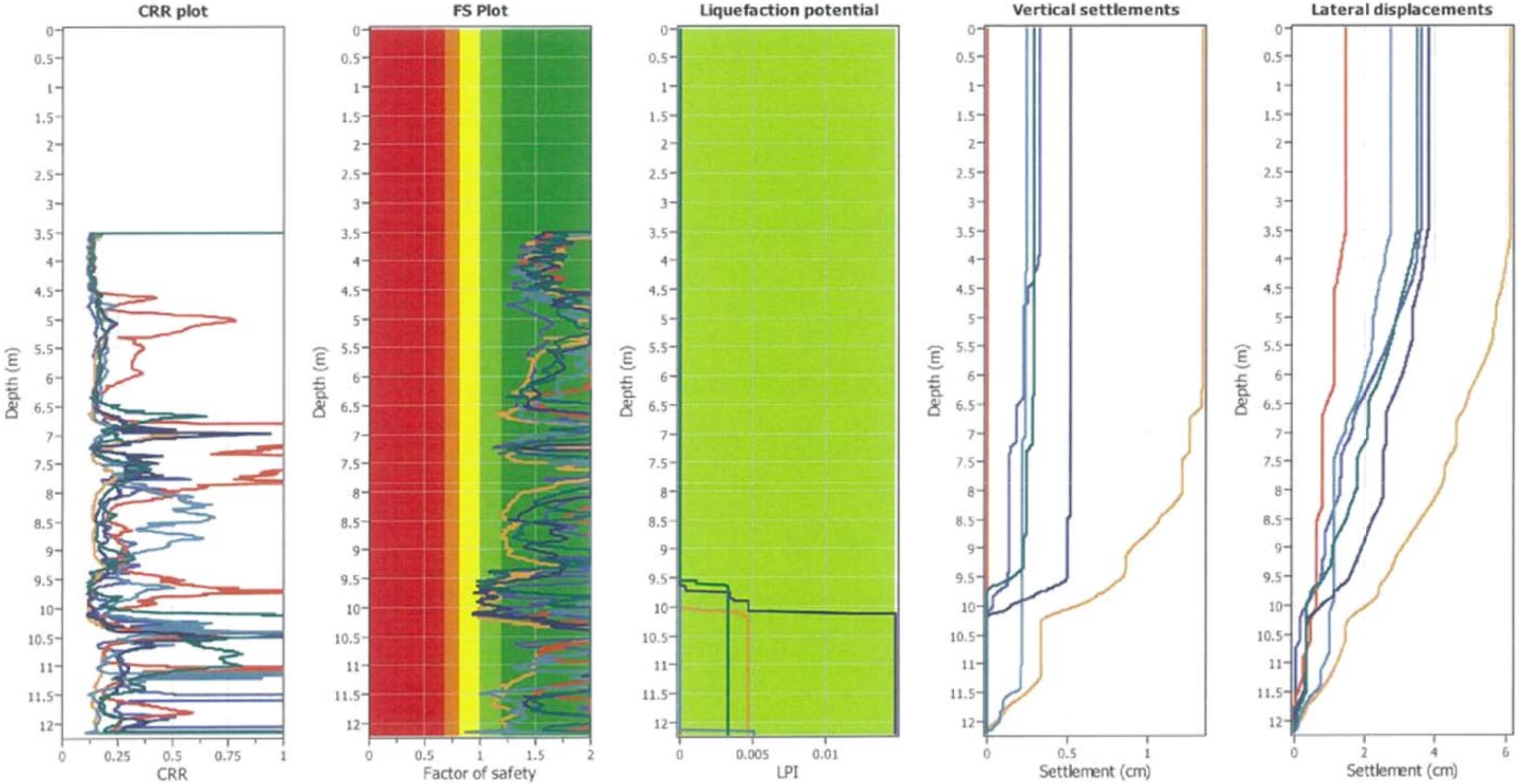
Project: 27 Shirley Road, Shirley

Overlay Intermediate Results



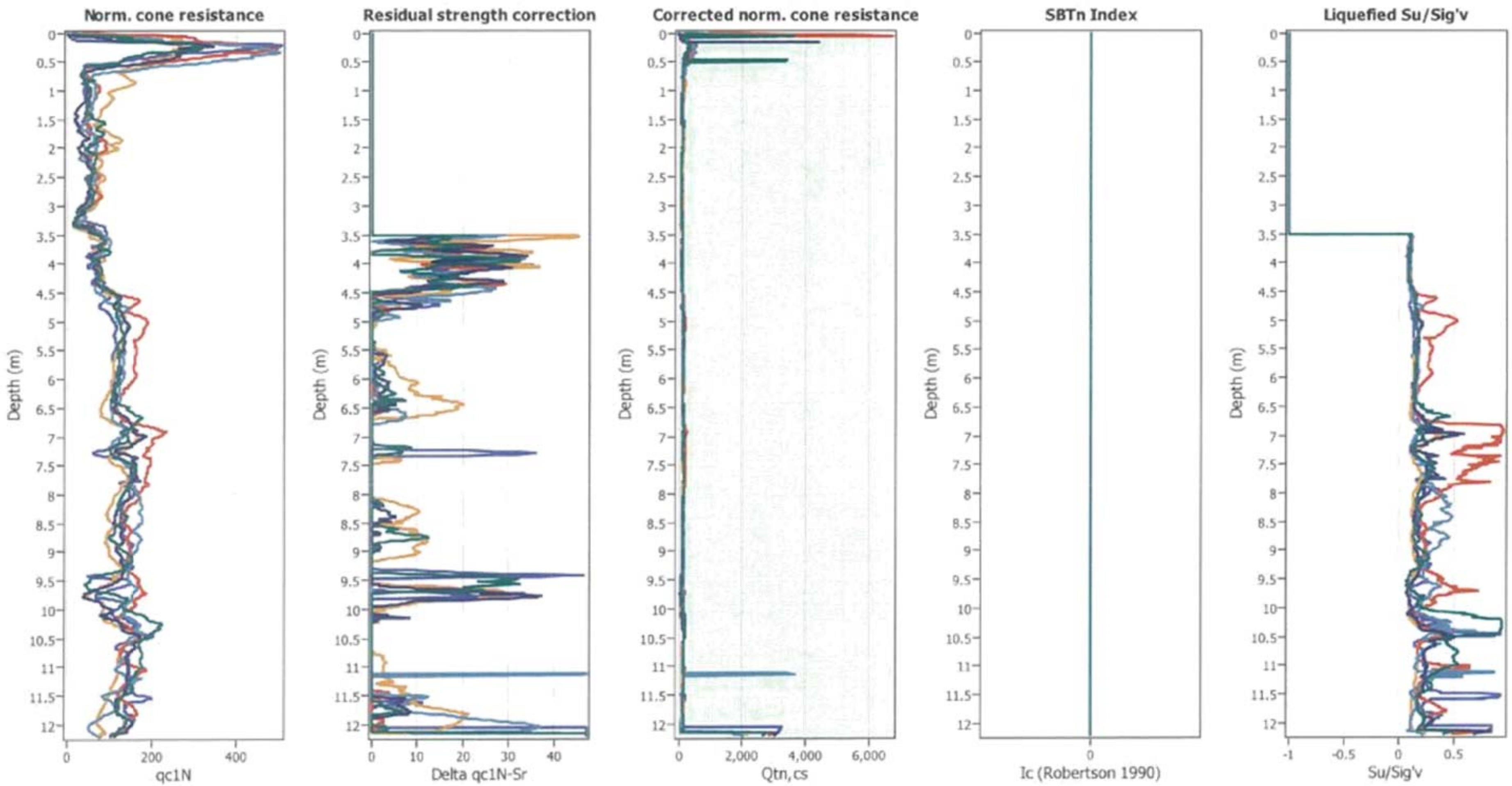
Project: 27 Shirley Road, Shirley

Overlay Cyclic Liquefaction Plots



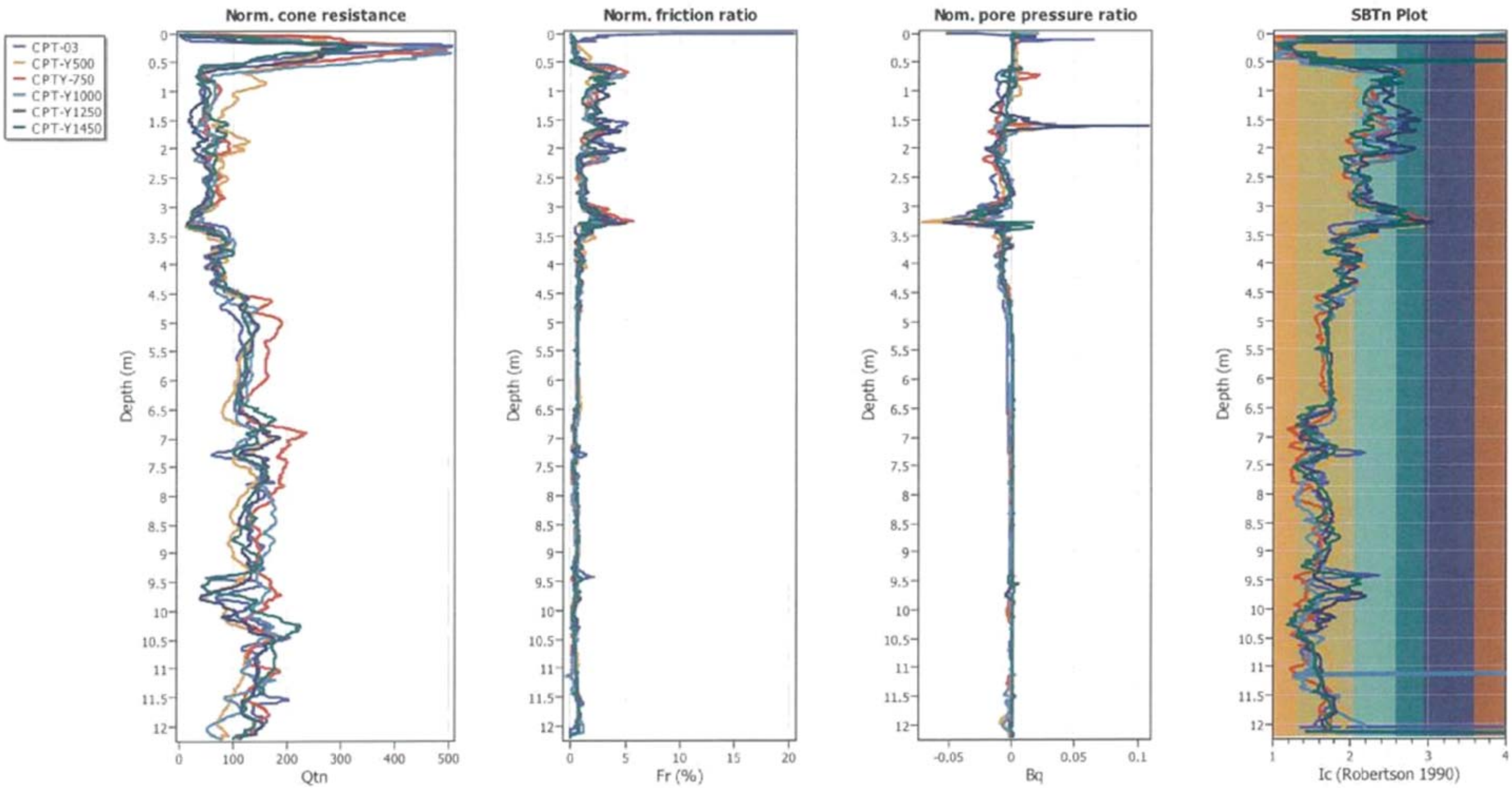
Project: 27 Shirley Road, Shirley

Overlay Strength Loss Plots



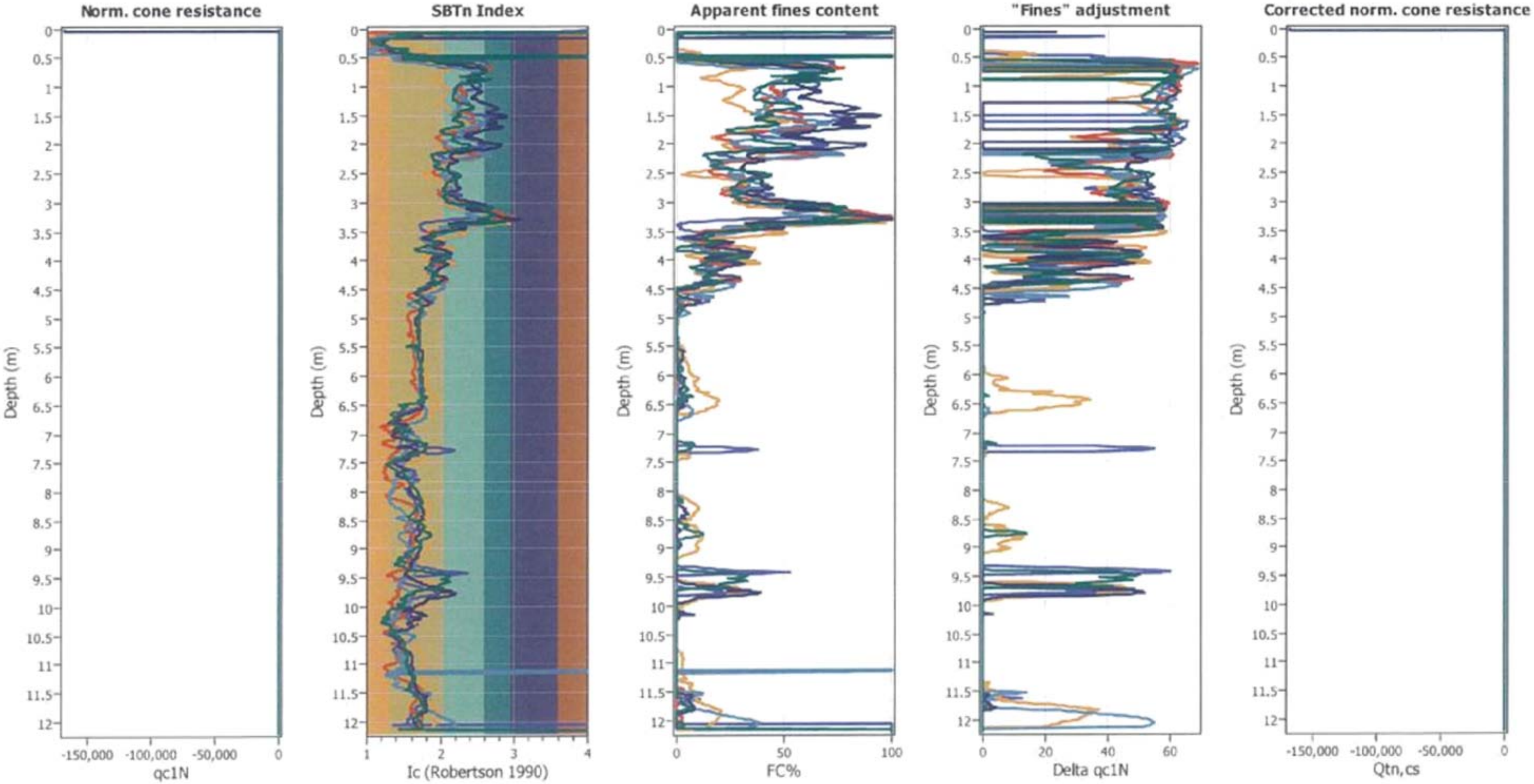
Project: 27 Shirley Road, Shirley

Overlay Normalized Plots



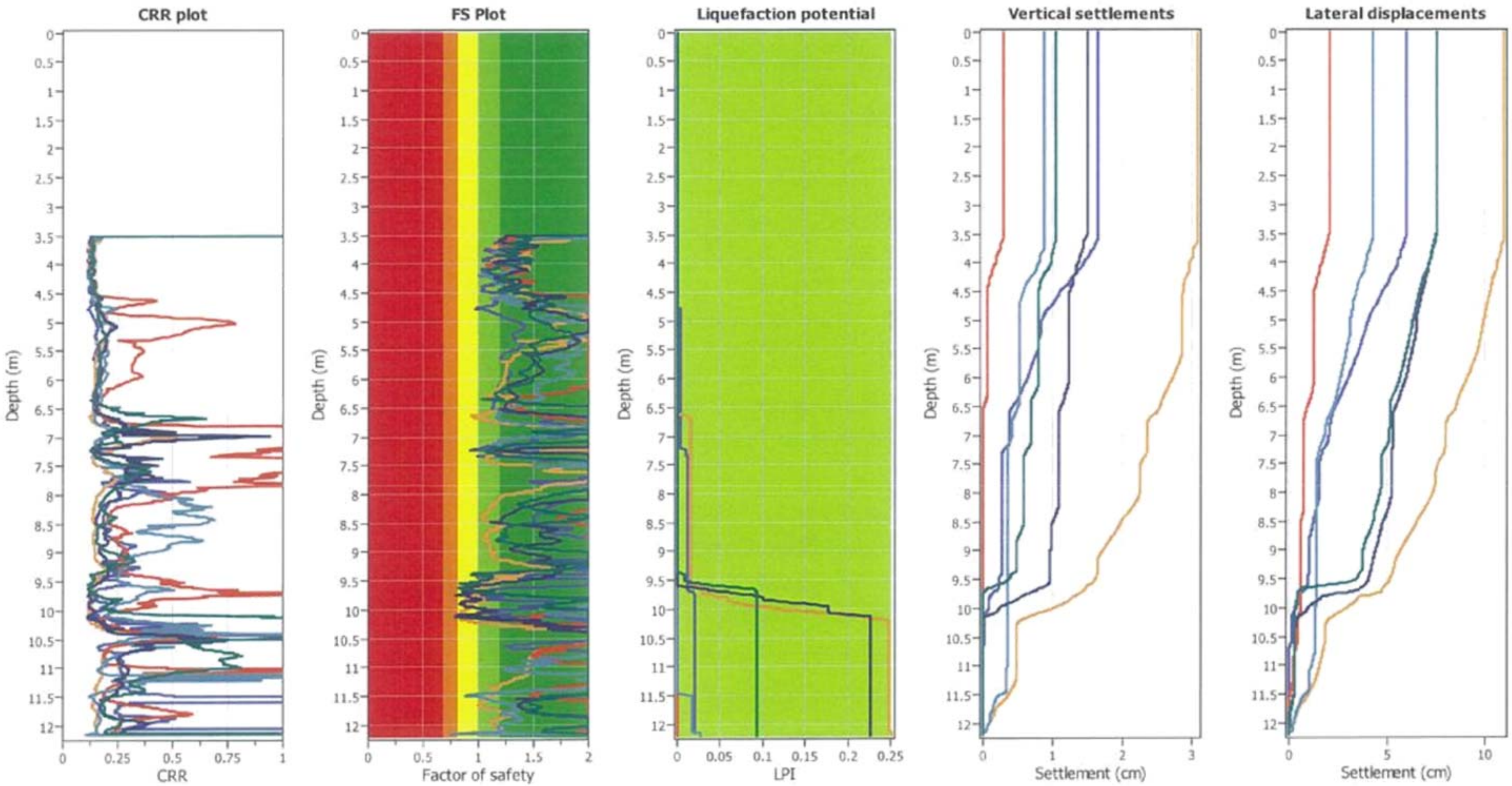
Project: 27 Shirley Road, Shirley

Overlay Intermediate Results



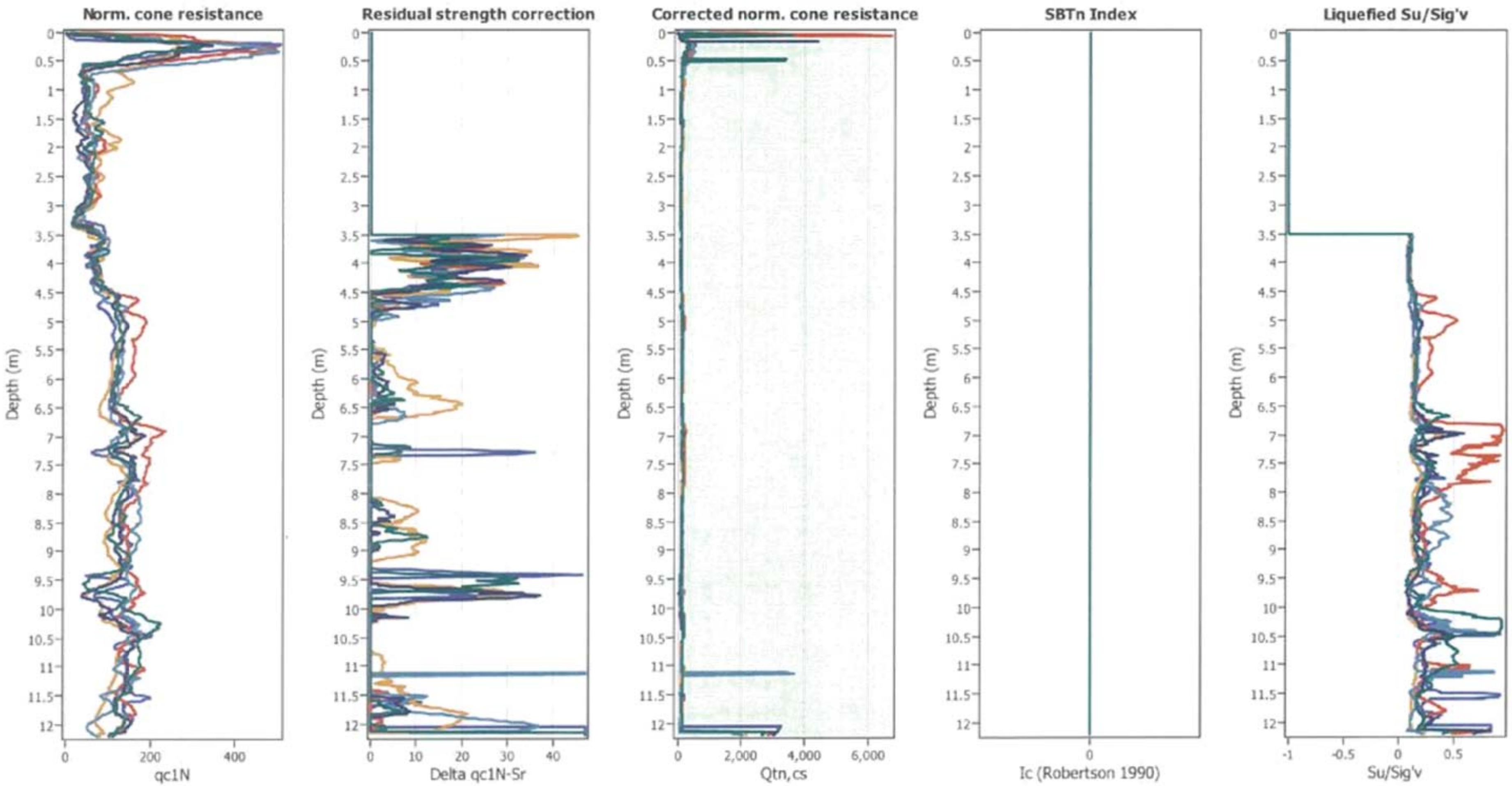
Project: 27 Shirley Road, Shirley

Overlay Cyclic Liquefaction Plots



Project: 27 Shirley Road, Shirley

Overlay Strength Loss Plots



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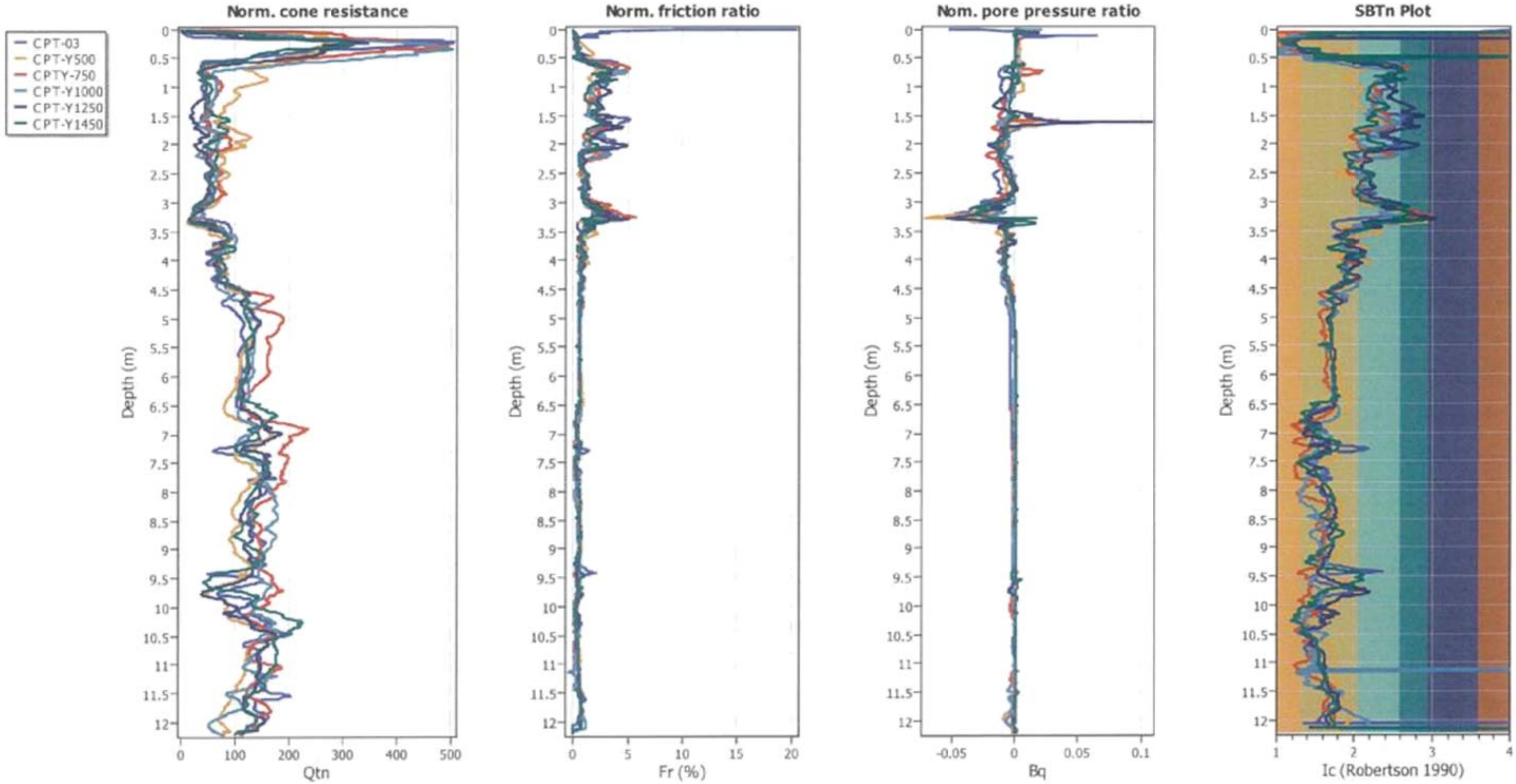
Project: 27 Shirley Road, Shirley

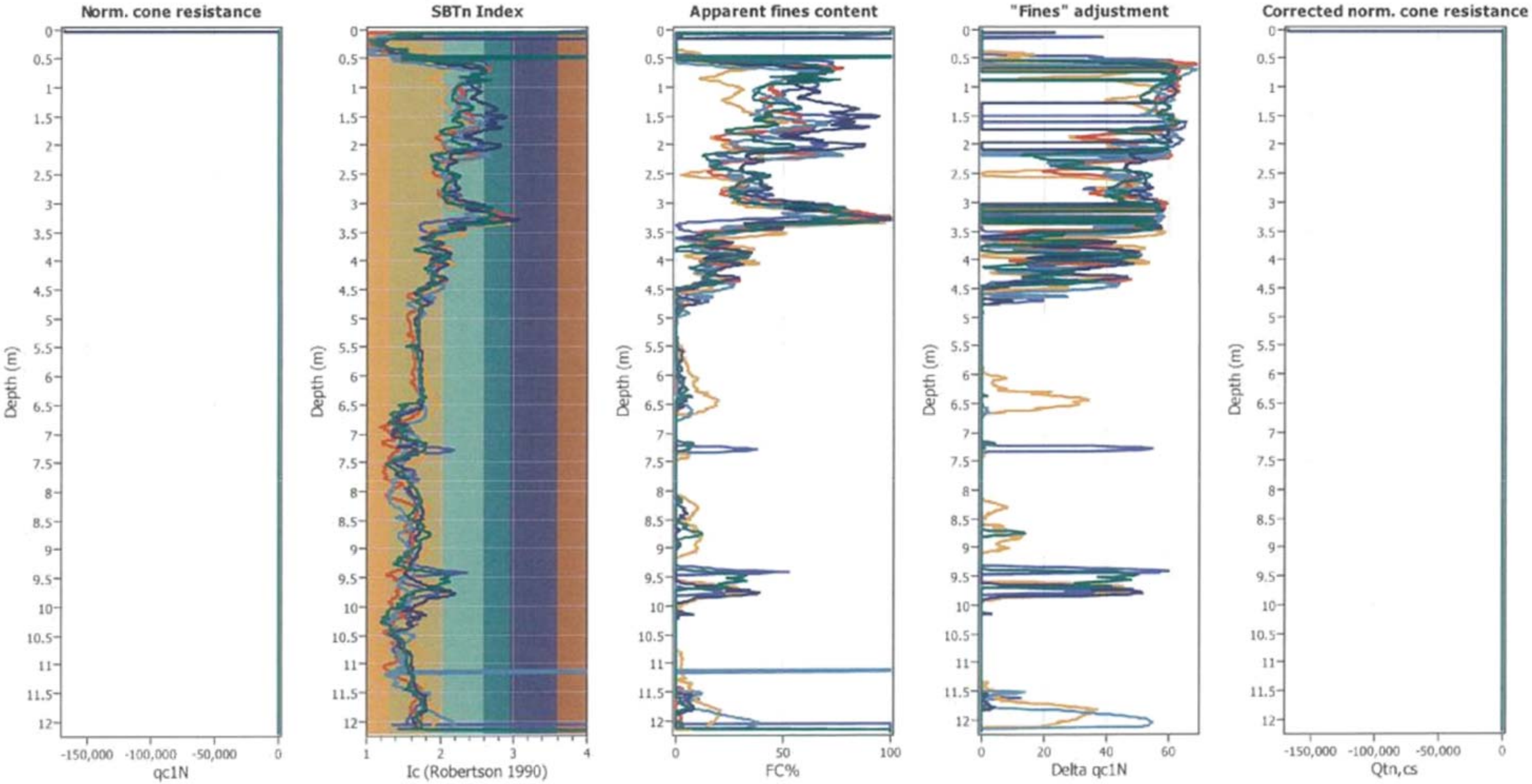
Overlay Normalized Plots



Project file: I:\Documents\HFoy Documents\Willowlea post construction Testing\CPTY.3 ULS.clq

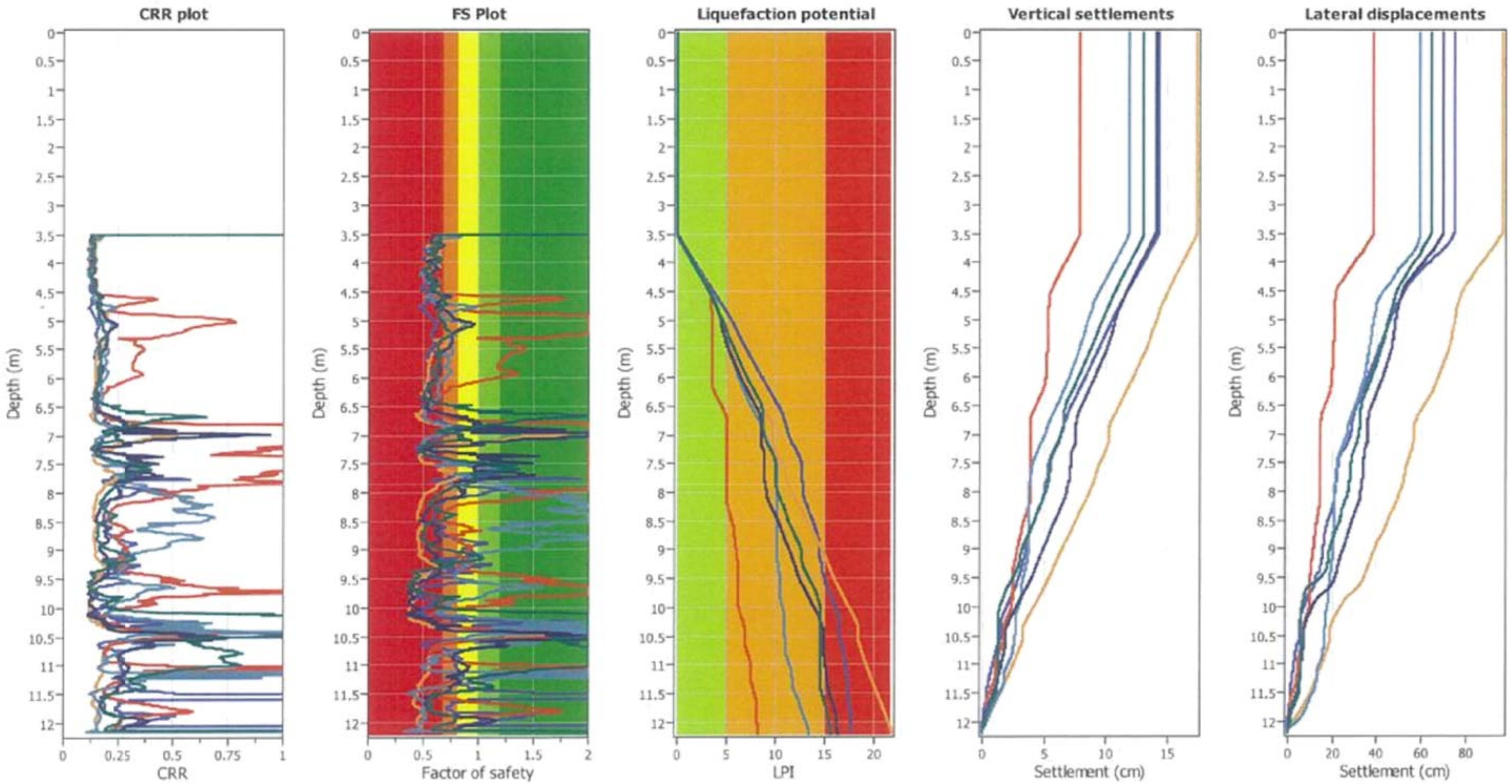
Project: 27 Shirley Road, Shirley

Overlay Intermediate Results



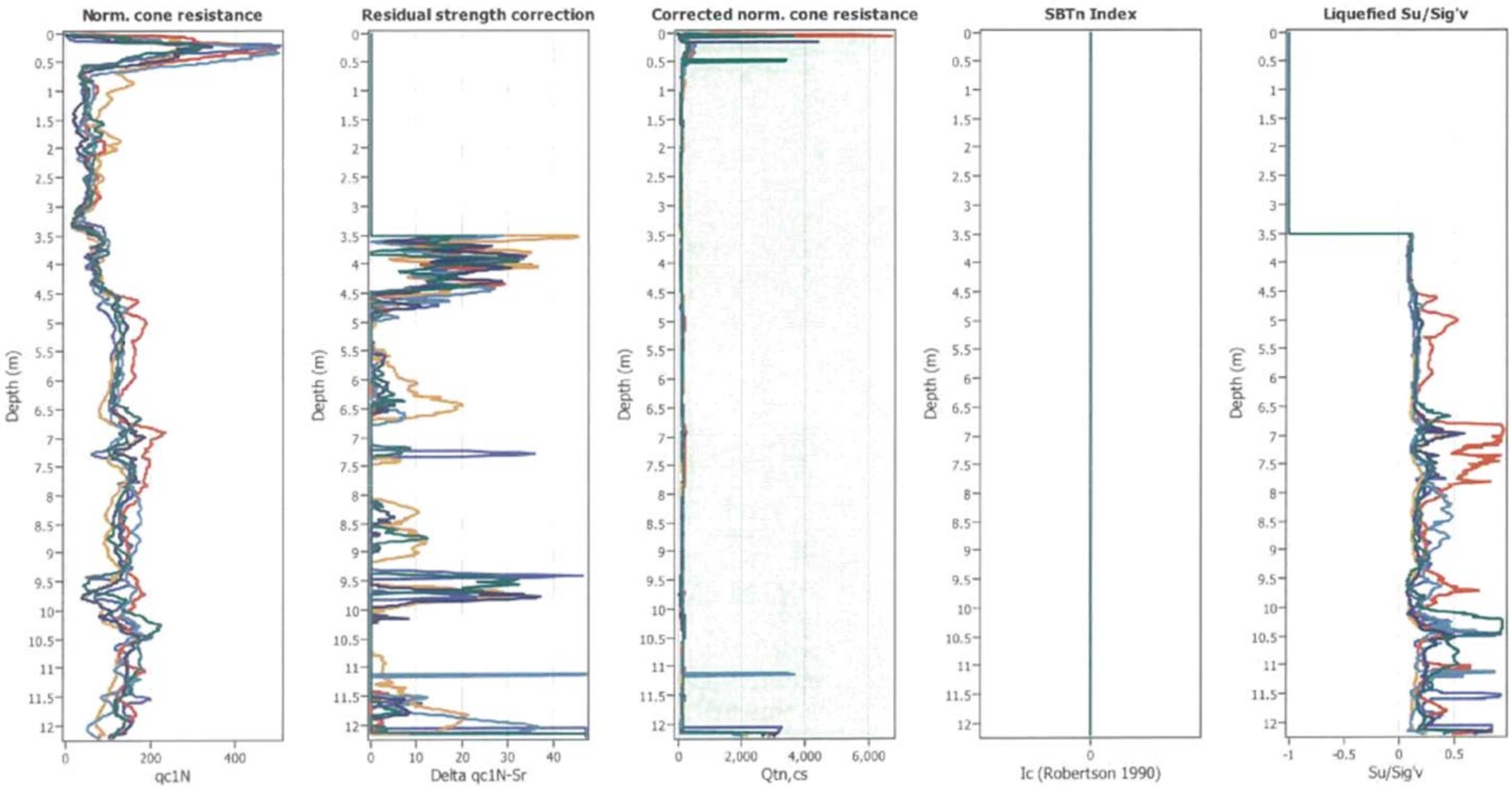
Project: 27 Shirley Road, Shirley

Overlay Cyclic Liquefaction Plots



Project: 27 Shirley Road, Shirley

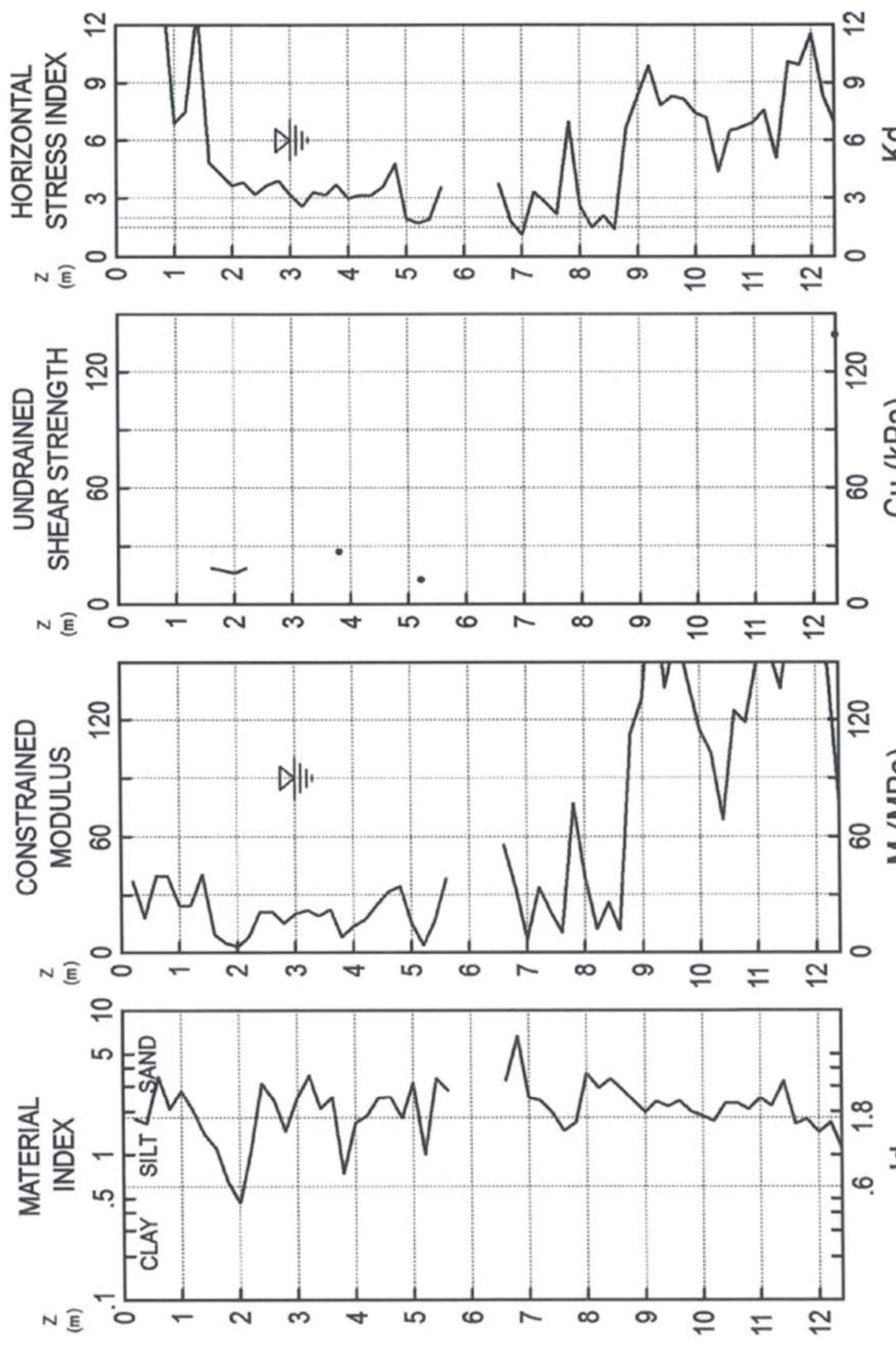
Overlay Strength Loss Plots



Appendix K – Research Pre DSM DMT and sDMT Results

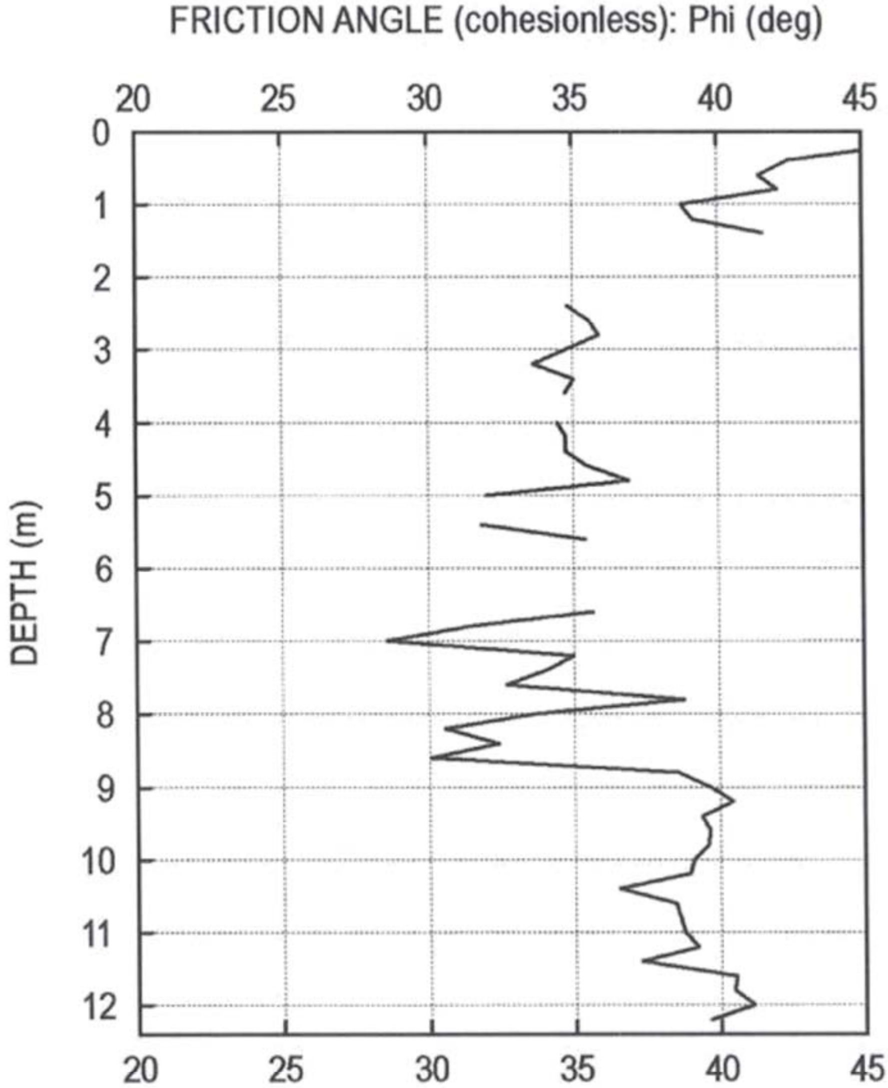
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**DMT-01**
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015

DILATOMETER TEST (D M T)



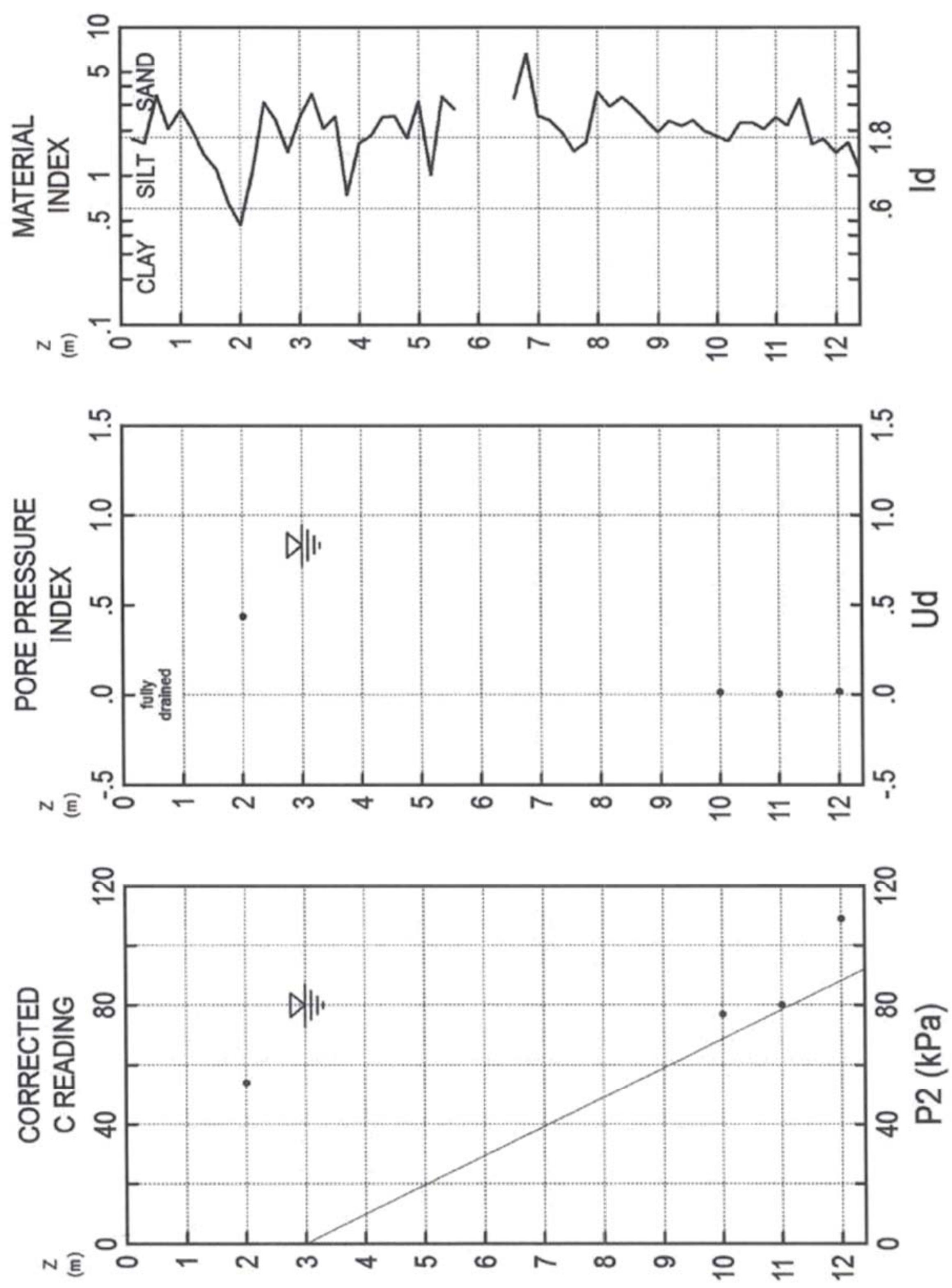
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	DMT-01
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



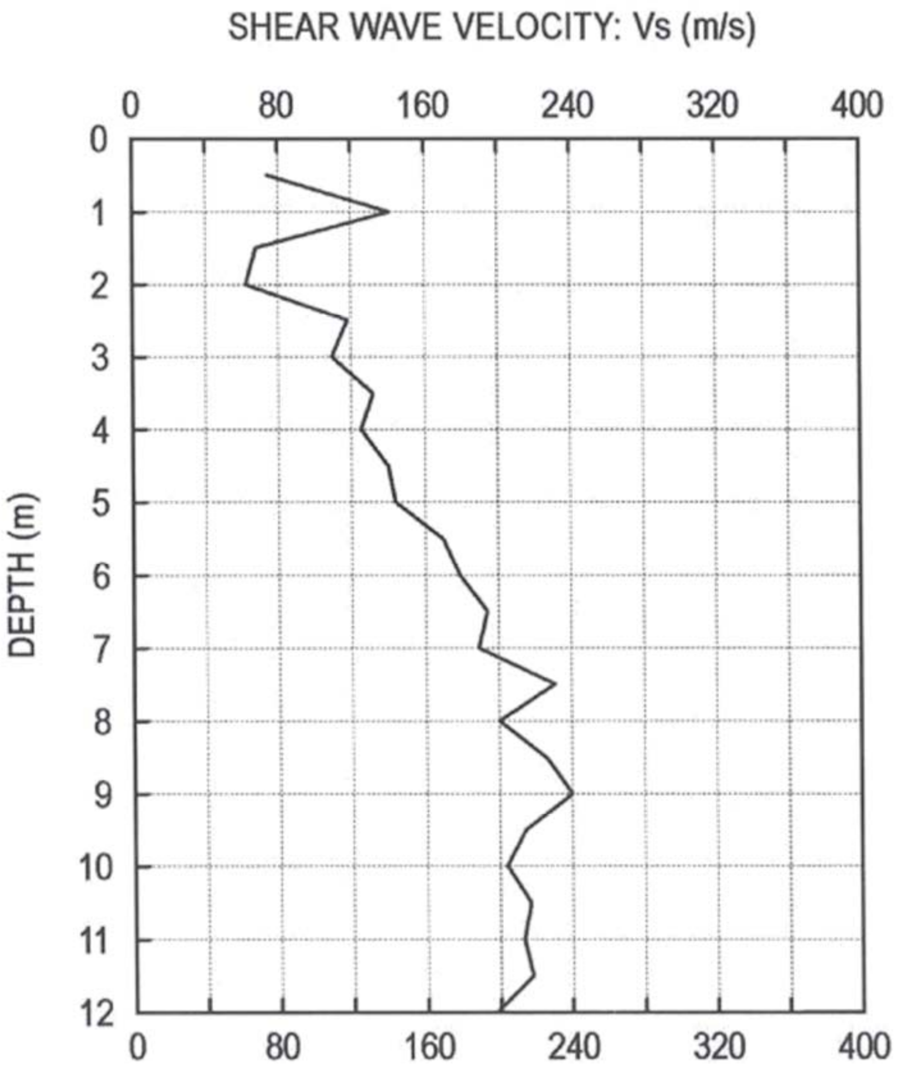
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**DMT-01**
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015

DILATOMETER TEST (D M T)



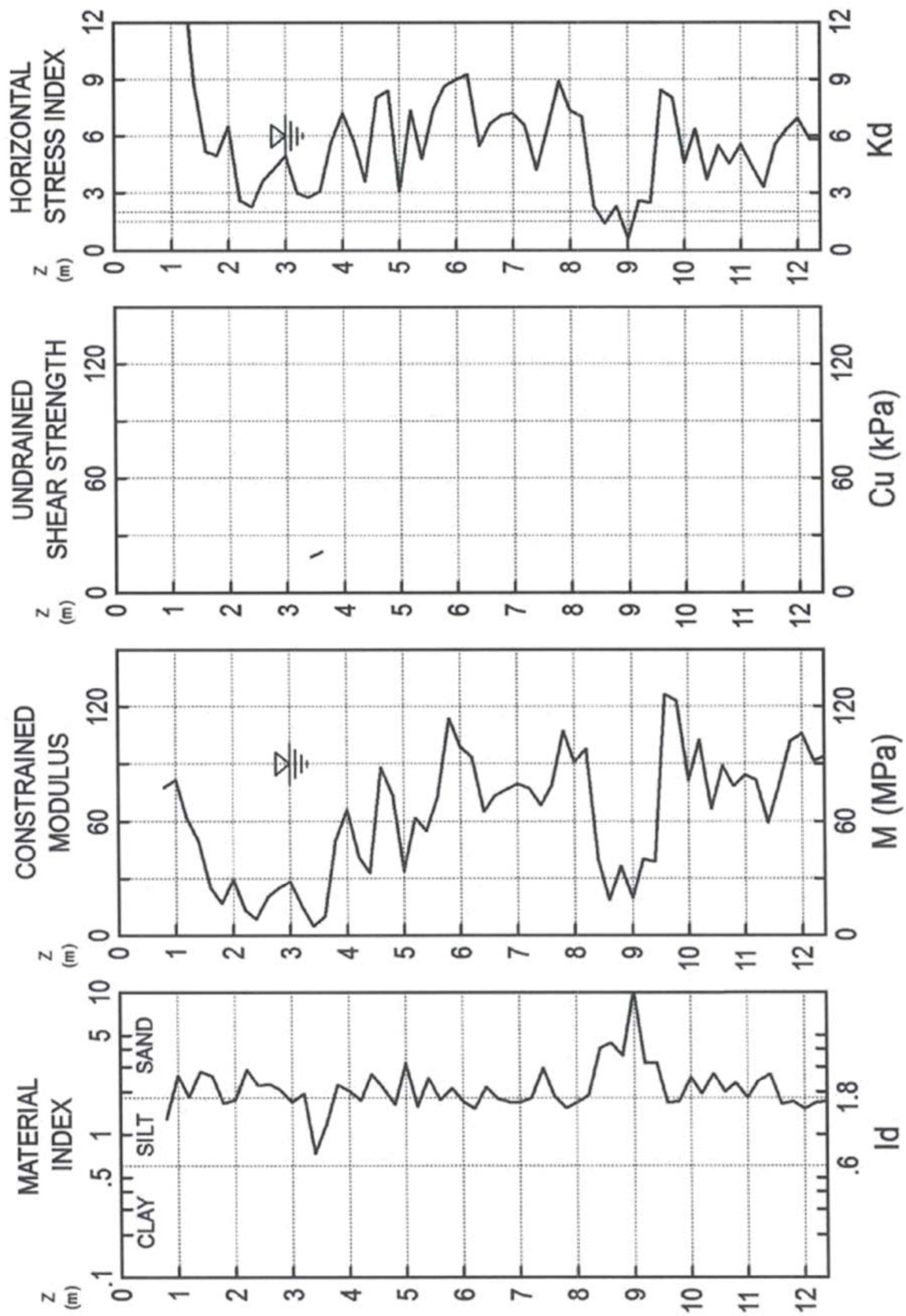
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**VS-01**
		10 SEP 2015



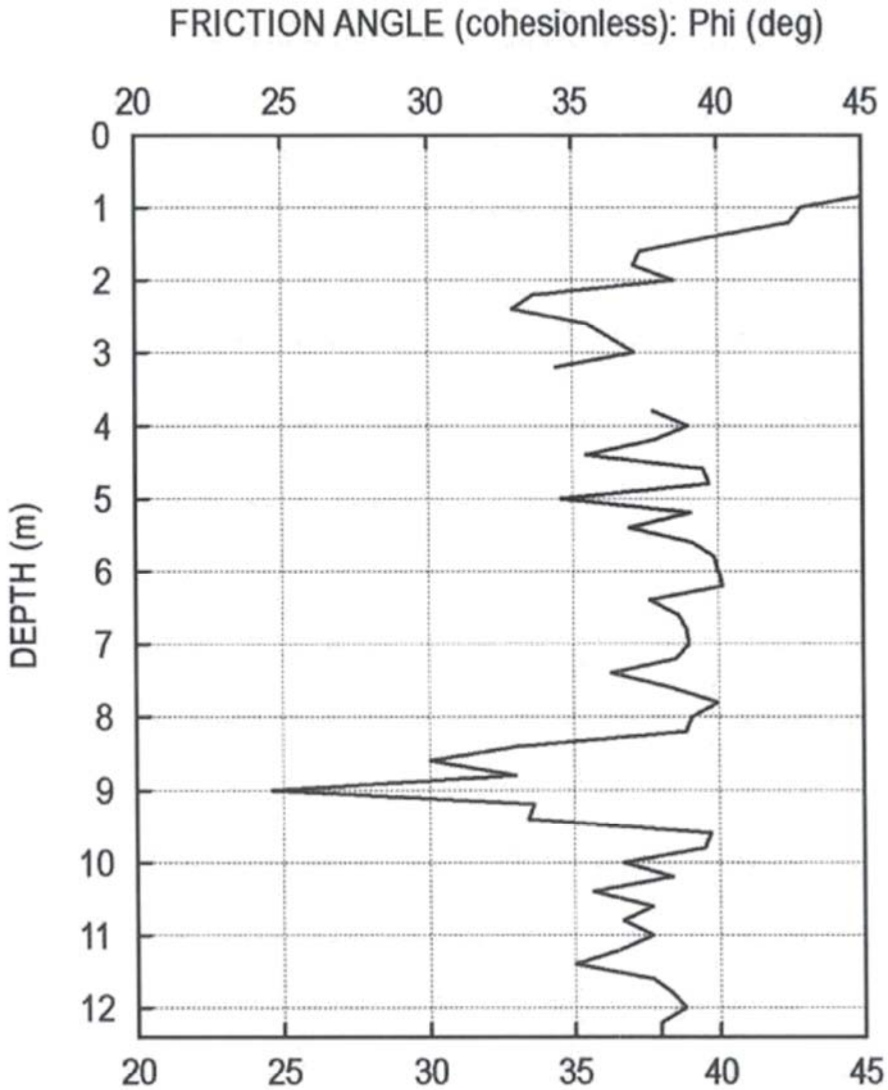
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**DMT-02**
INTERPRETED GEOTECHNICAL PARAMETERS		11 SEP 2015

DILATOMETER TEST (D M T)



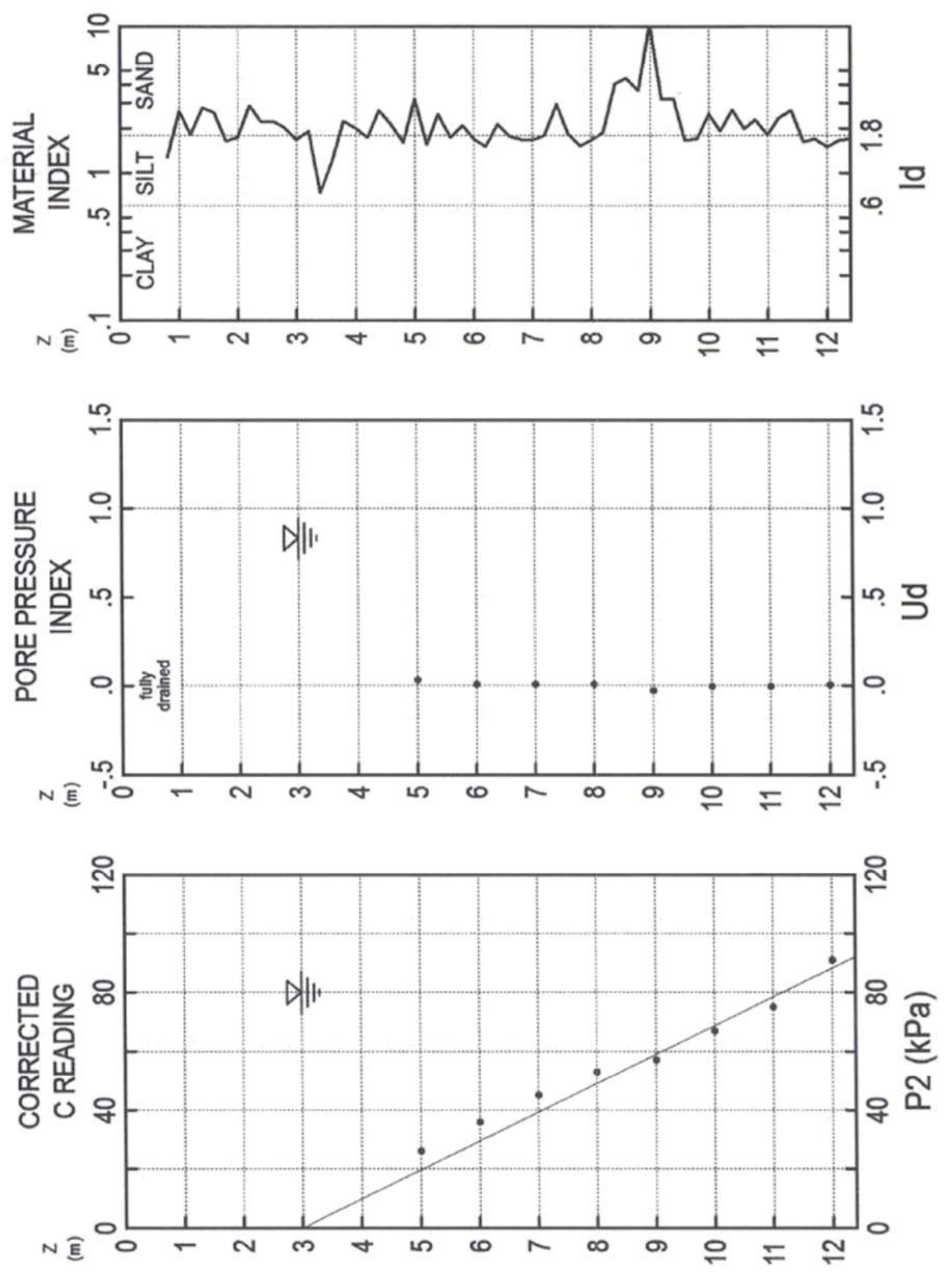
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**DMT-02**
INTERPRETED GEOTECHNICAL PARAMETERS		11 SEP 2015



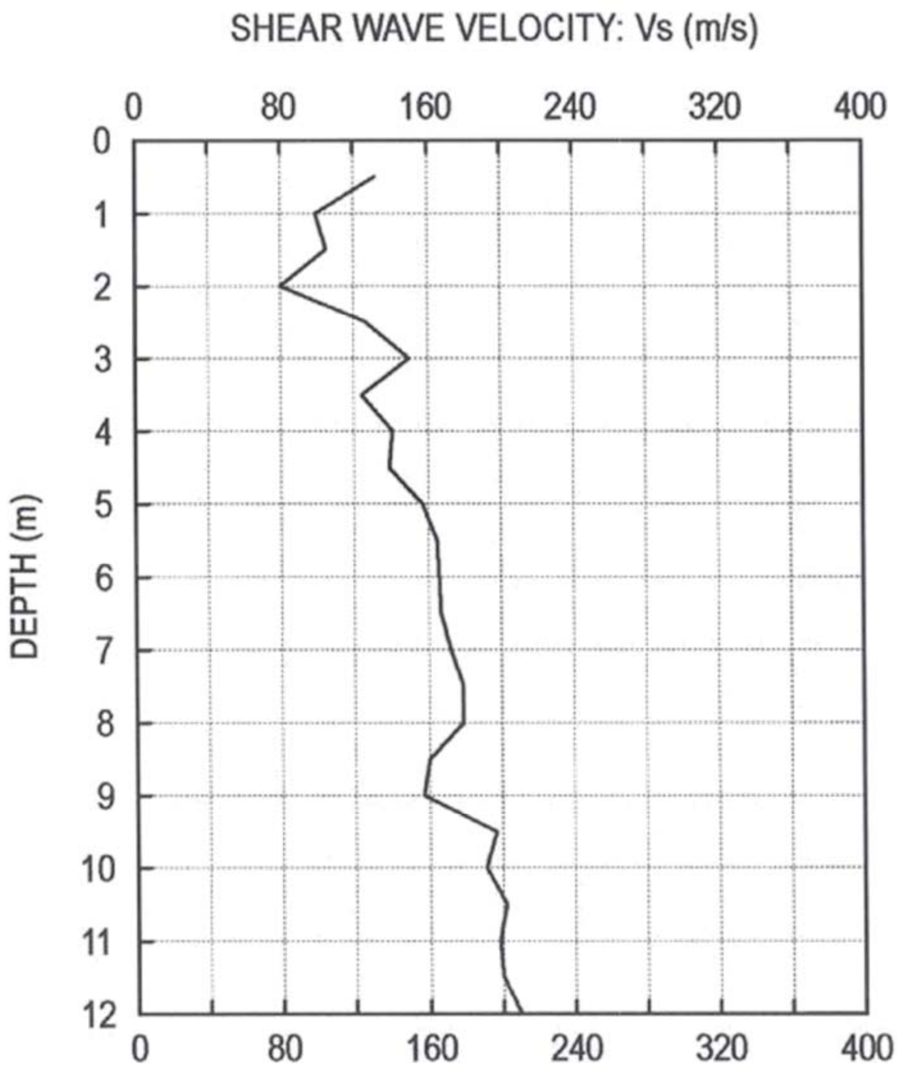
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	DMT-02
INTERPRETED GEOTECHNICAL PARAMETERS		11 SEP 2015



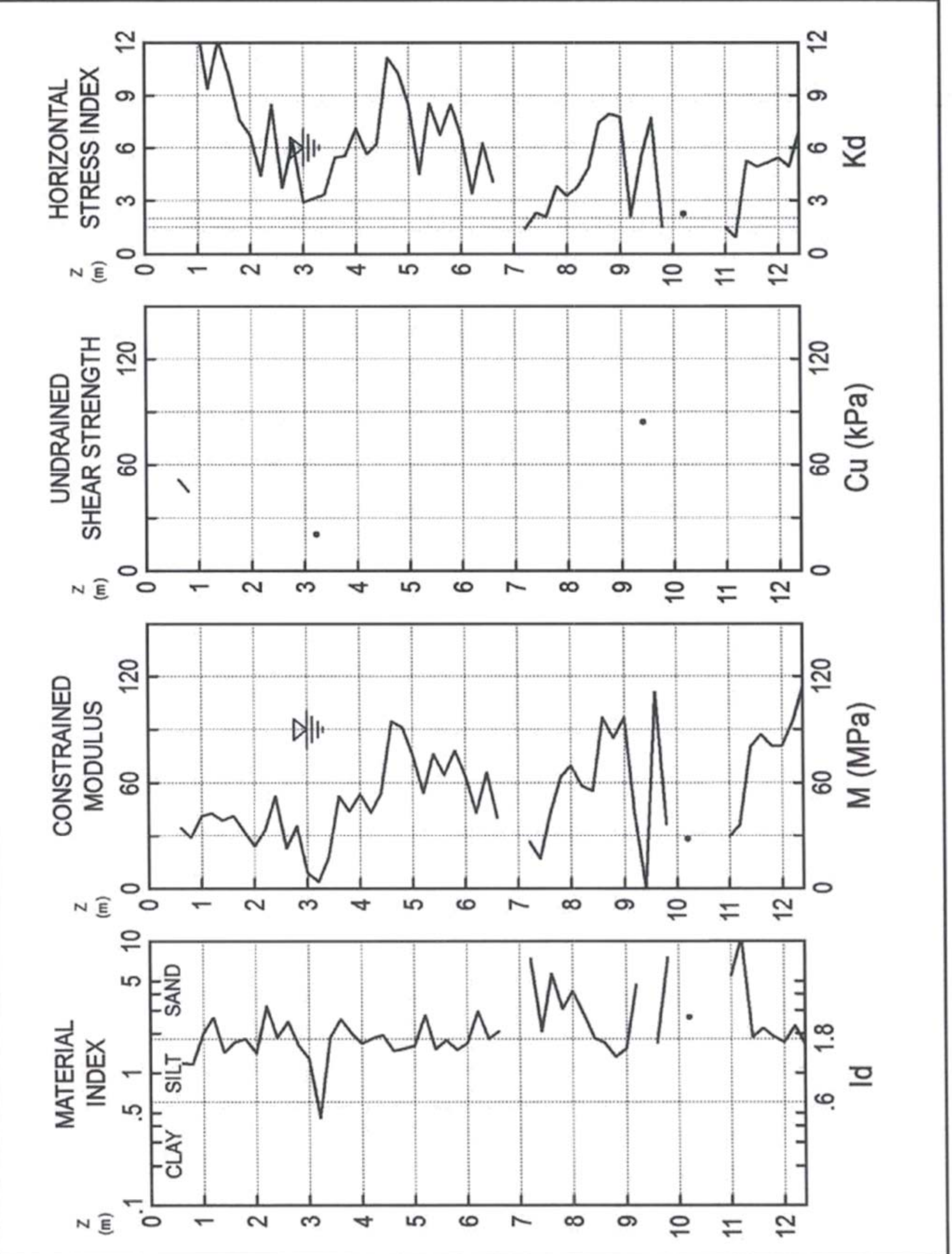
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	VS-02
		11 SEP 2015



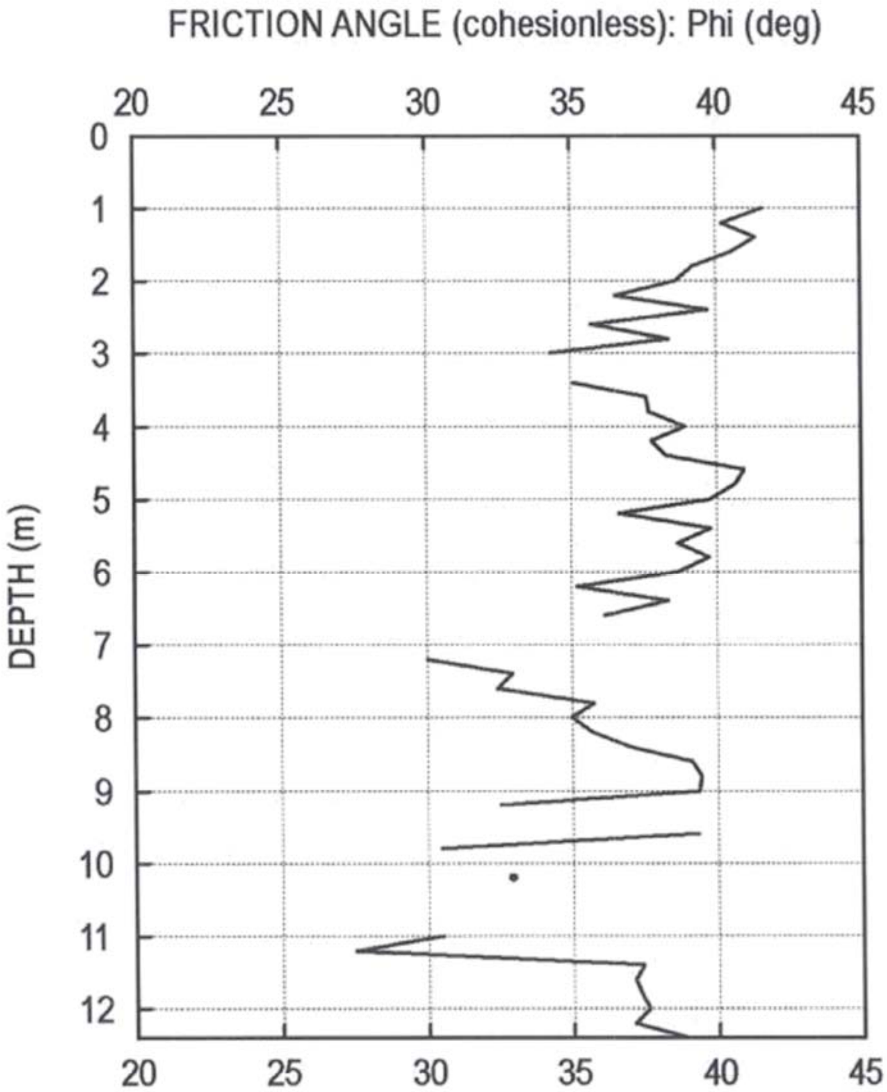
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**DMT-03**
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	DMT-03
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



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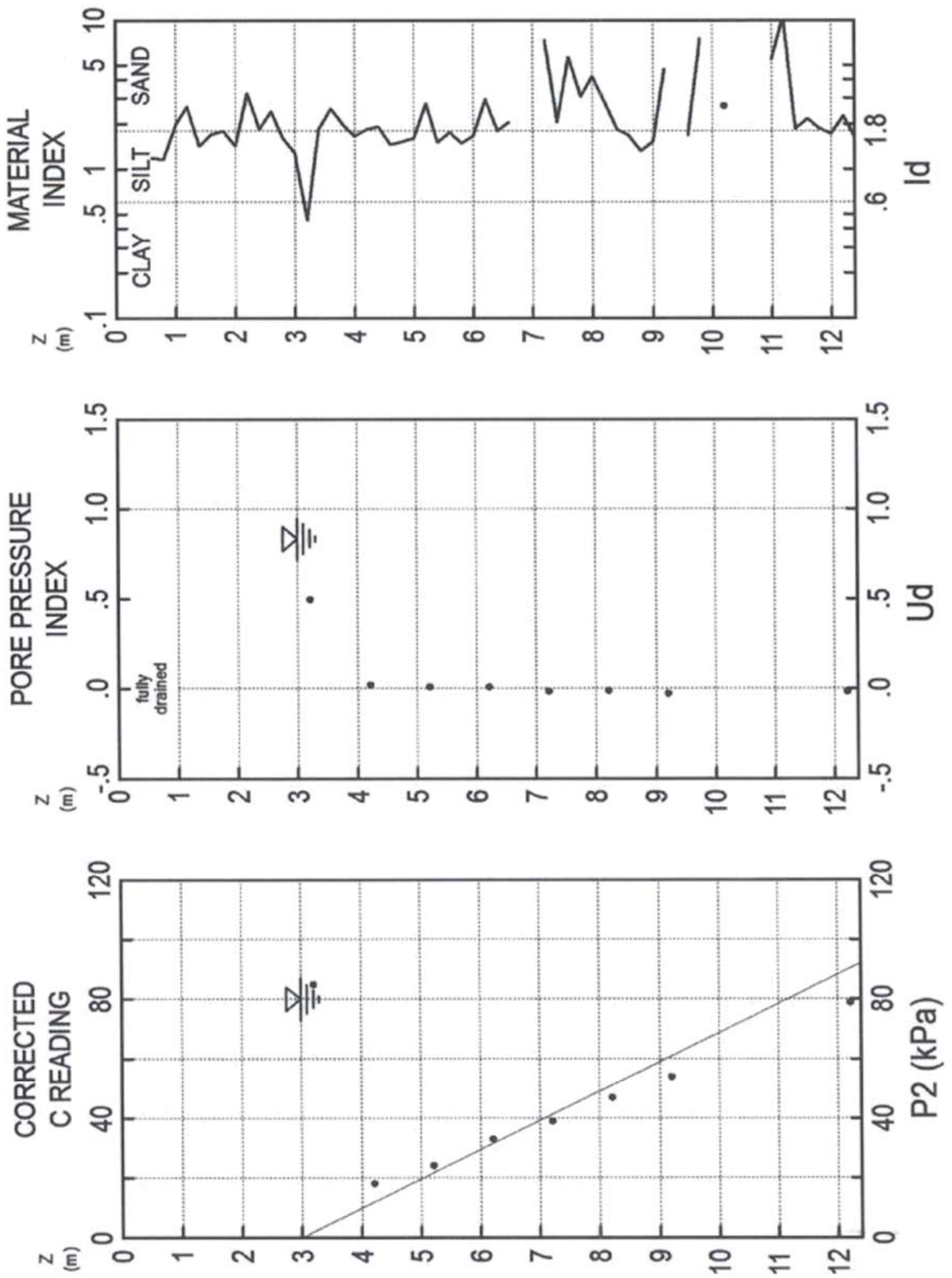
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**DMT-03**
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015

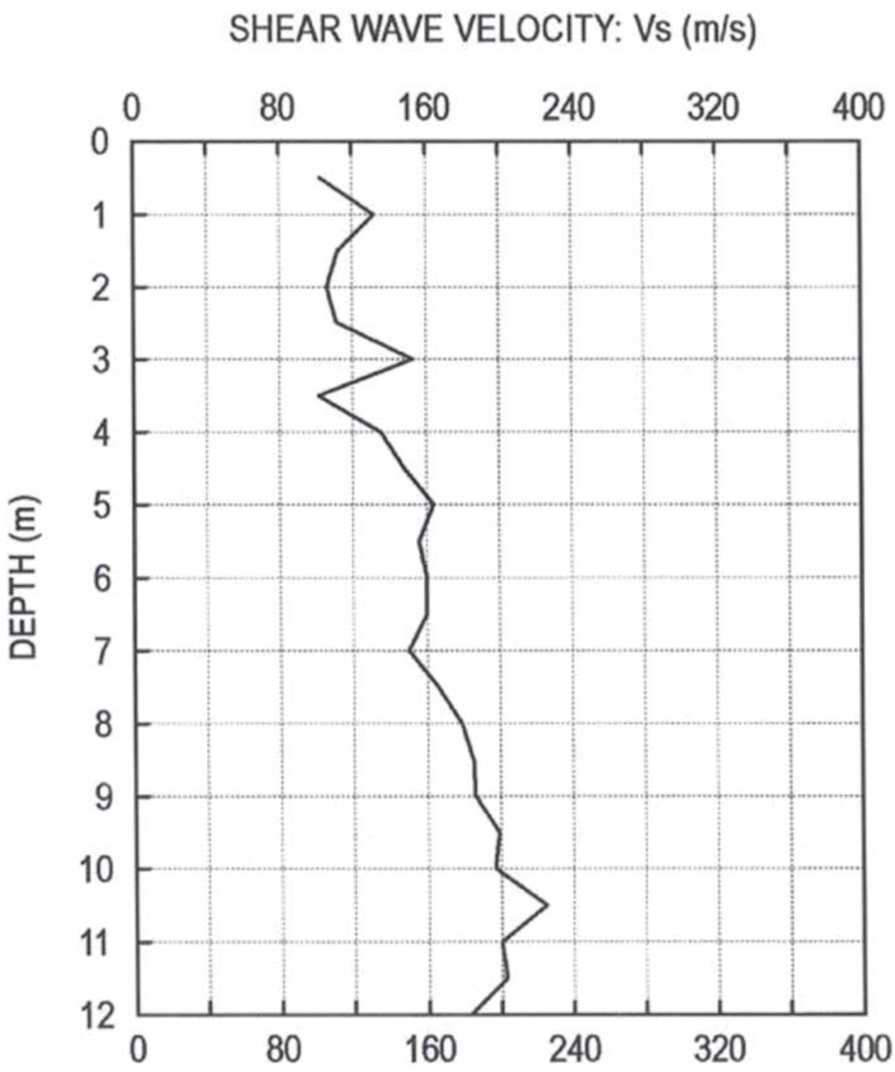
DILATOMETER TEST (DMT)



12

SEISMIC DILATOMETER TEST (S D M T)

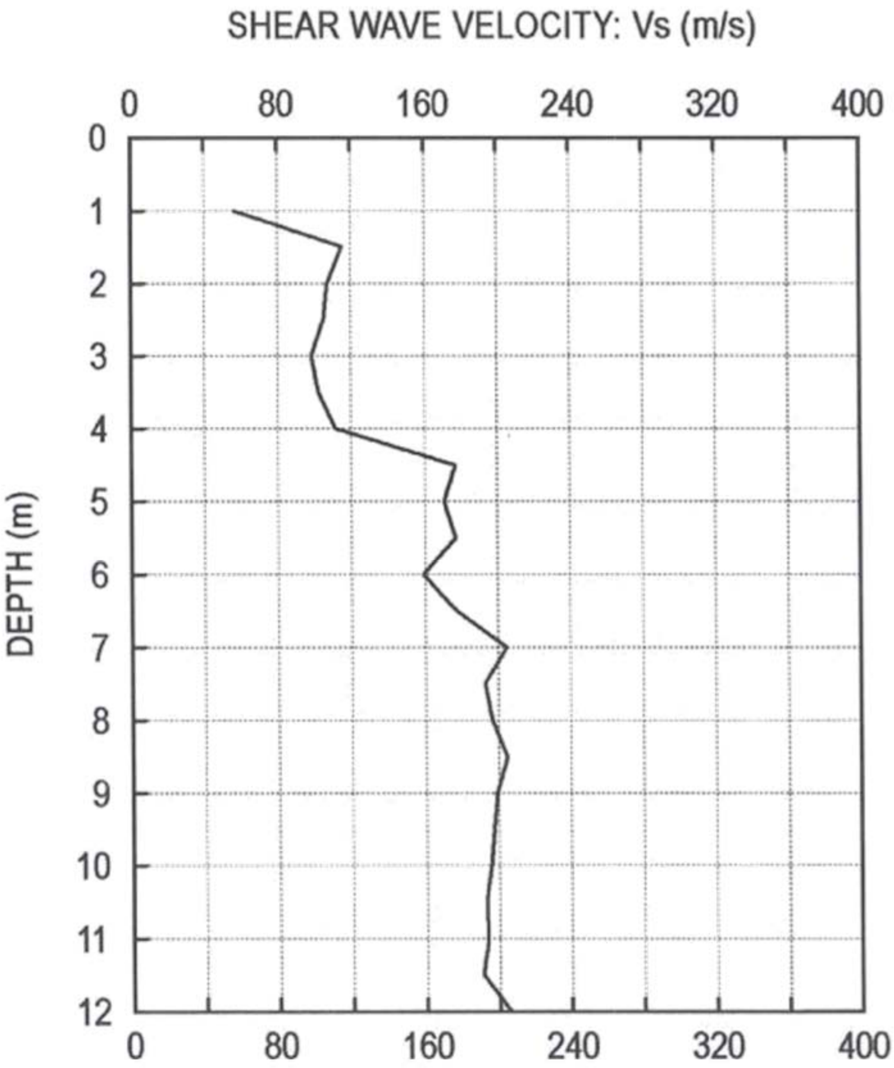
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	VS-03
		10 SEP 2015



13

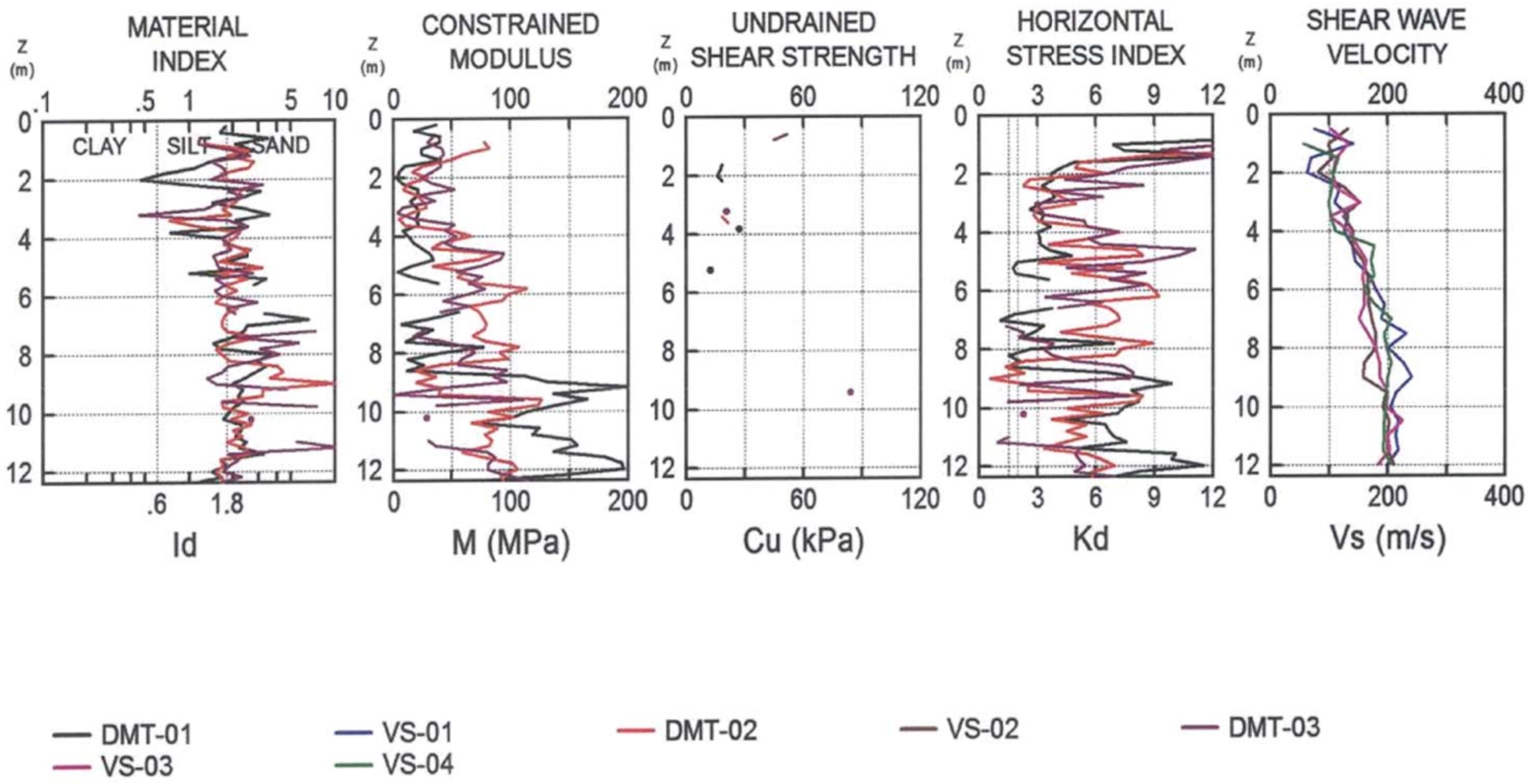
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	VS-04
		10 SEP 2015



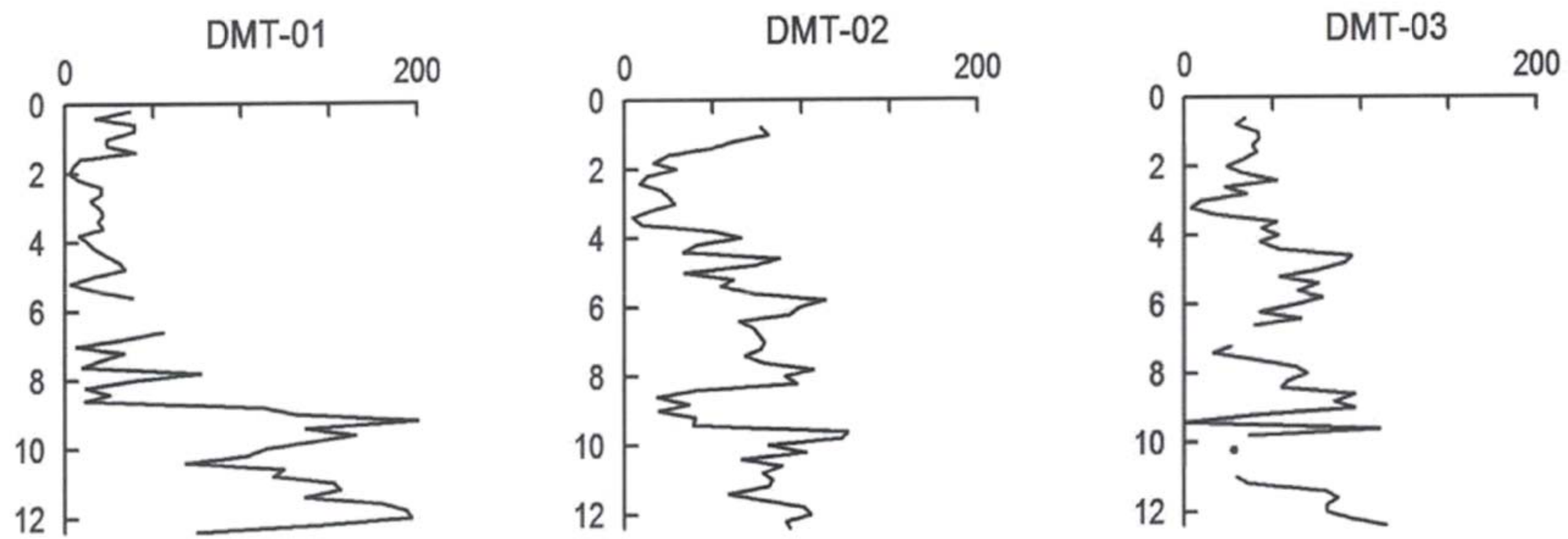
Ground Investigation
15-169

Hiways Geotechnical
27 Shirley Rd



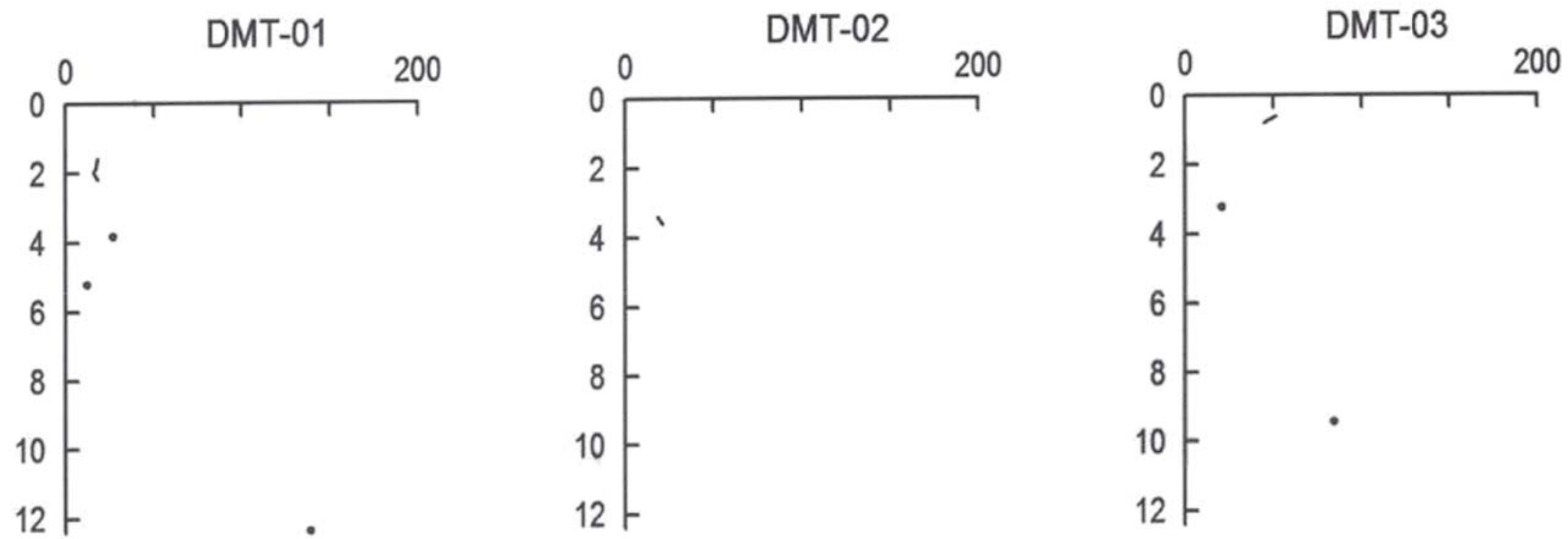
Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd

CROSS SECTION OF CONSTRAINED MODULUS M (MPa)



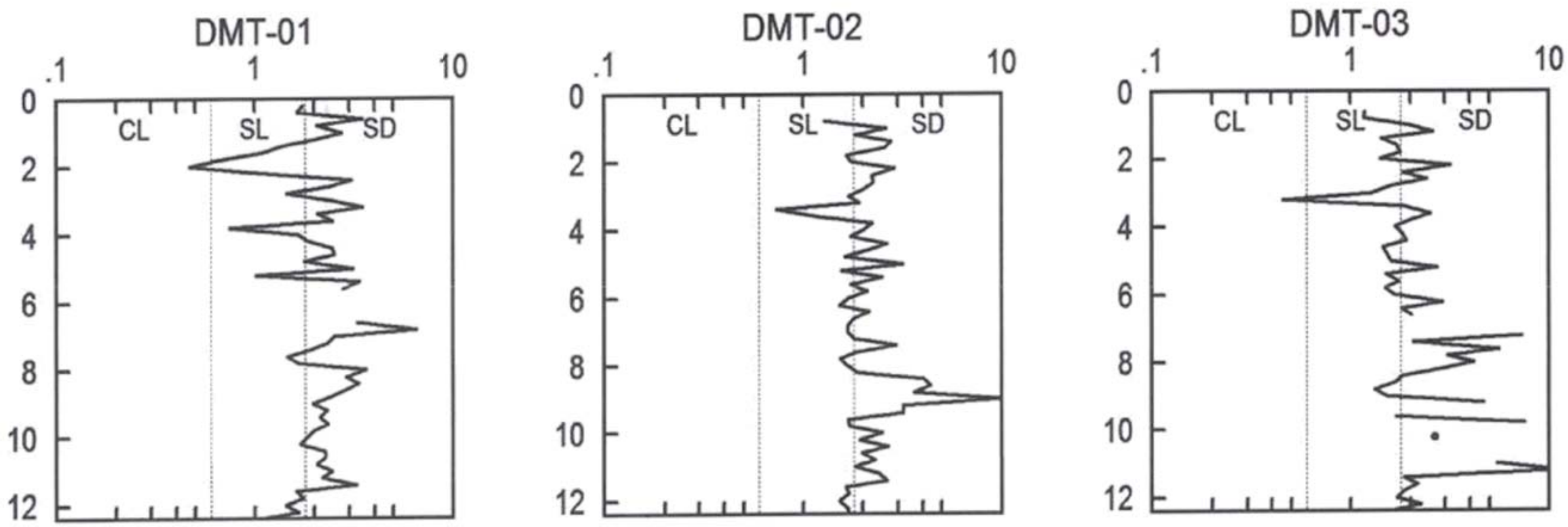
Ground Investigation | Hiways Geotechnical
15-169 | 27 Shirley Rd

CROSS SECTION OF UNDRAINED SHEAR STRENGTH Cu (kPa)



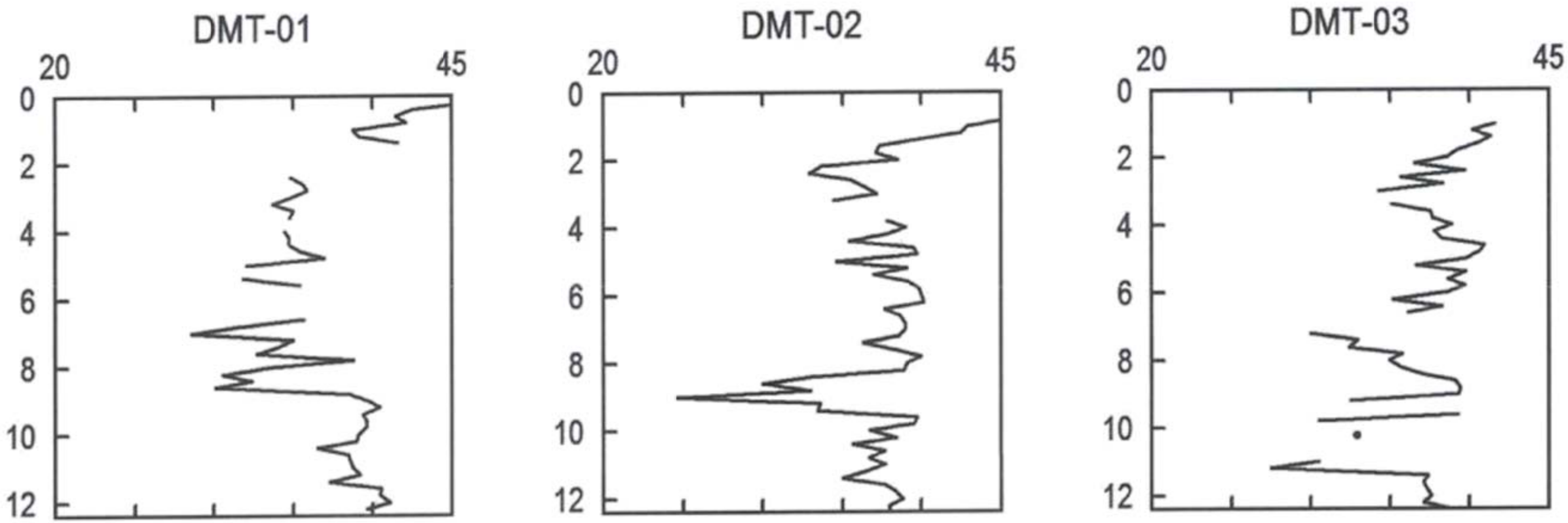
Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd

CROSS SECTION OF MATERIAL INDEX Id



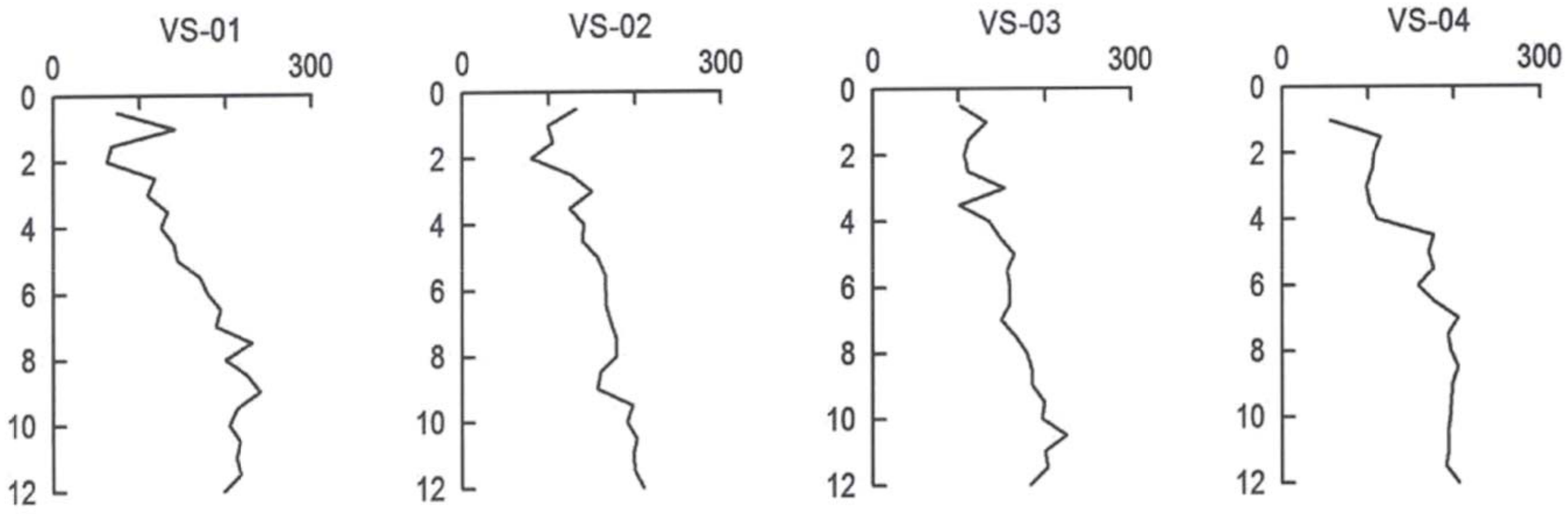
Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd

CROSS SECTION OF FRICTION ANGLE (cohesionless) Phi (deg)



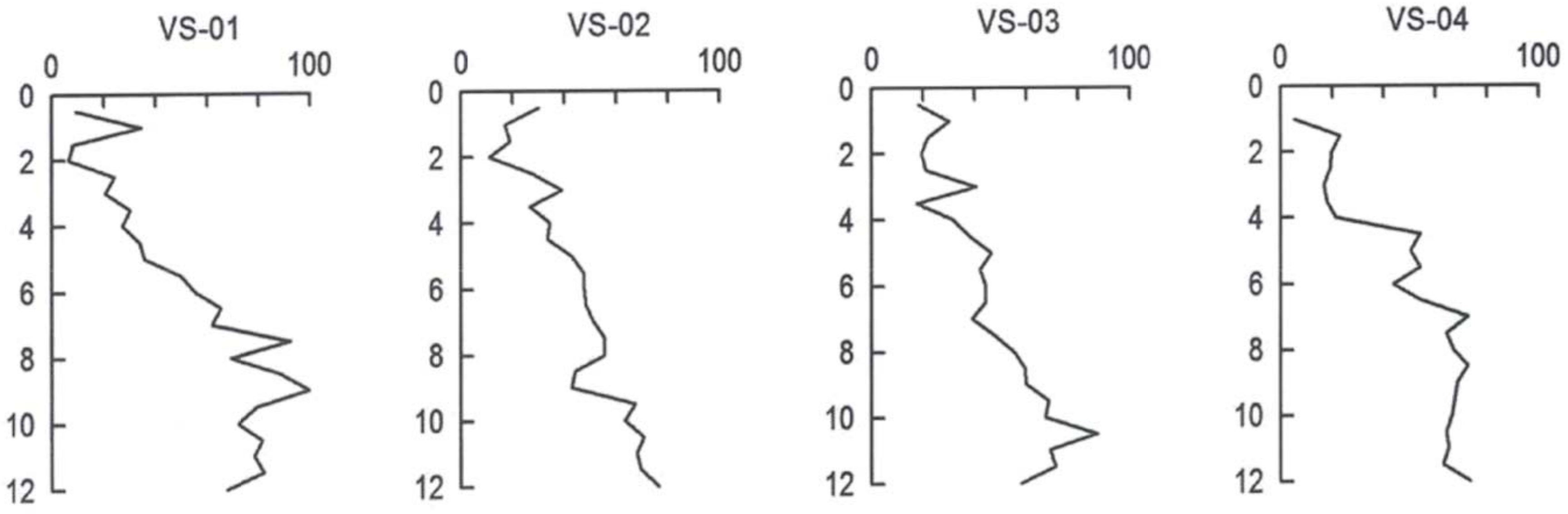
Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd

CROSS SECTION OF SHEAR WAVE VELOCITY Vs (m/s)

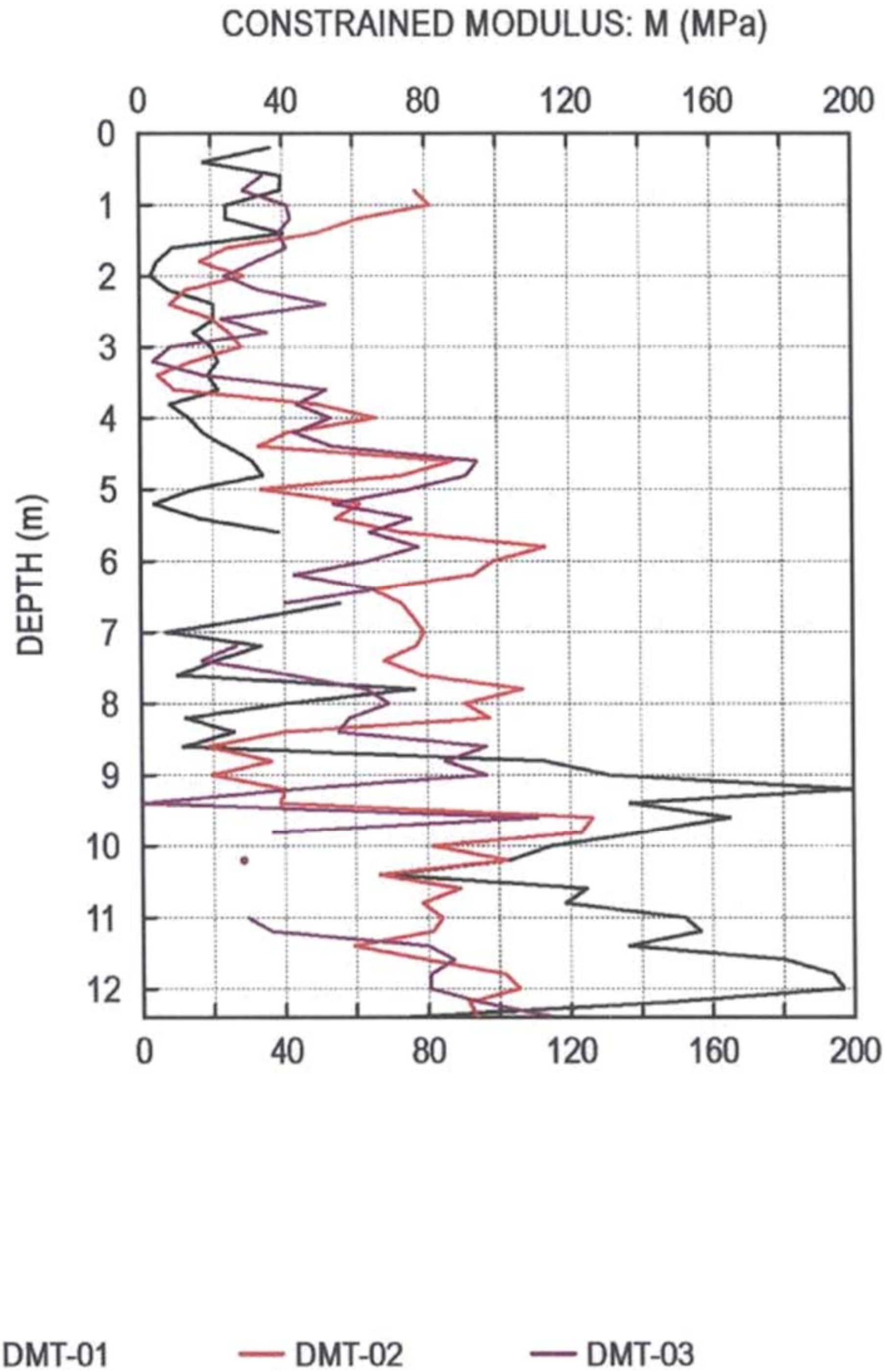


Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd

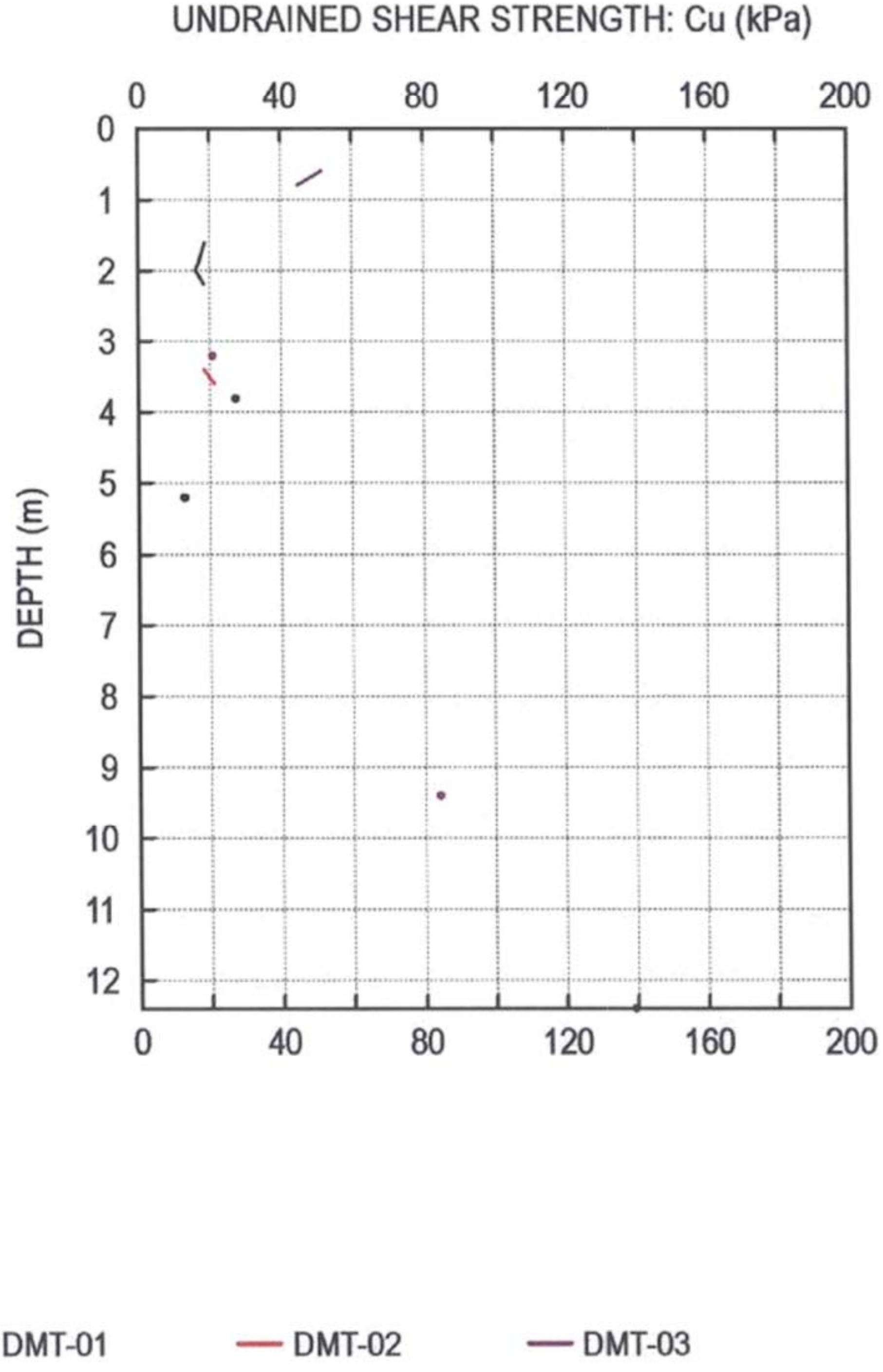
CROSS SECTION OF MAX SHEAR MODULUS Go (MPa)



Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd
SUPERIMPOSED TEST RESULTS	



Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd
SUPERIMPOSED TEST RESULTS	



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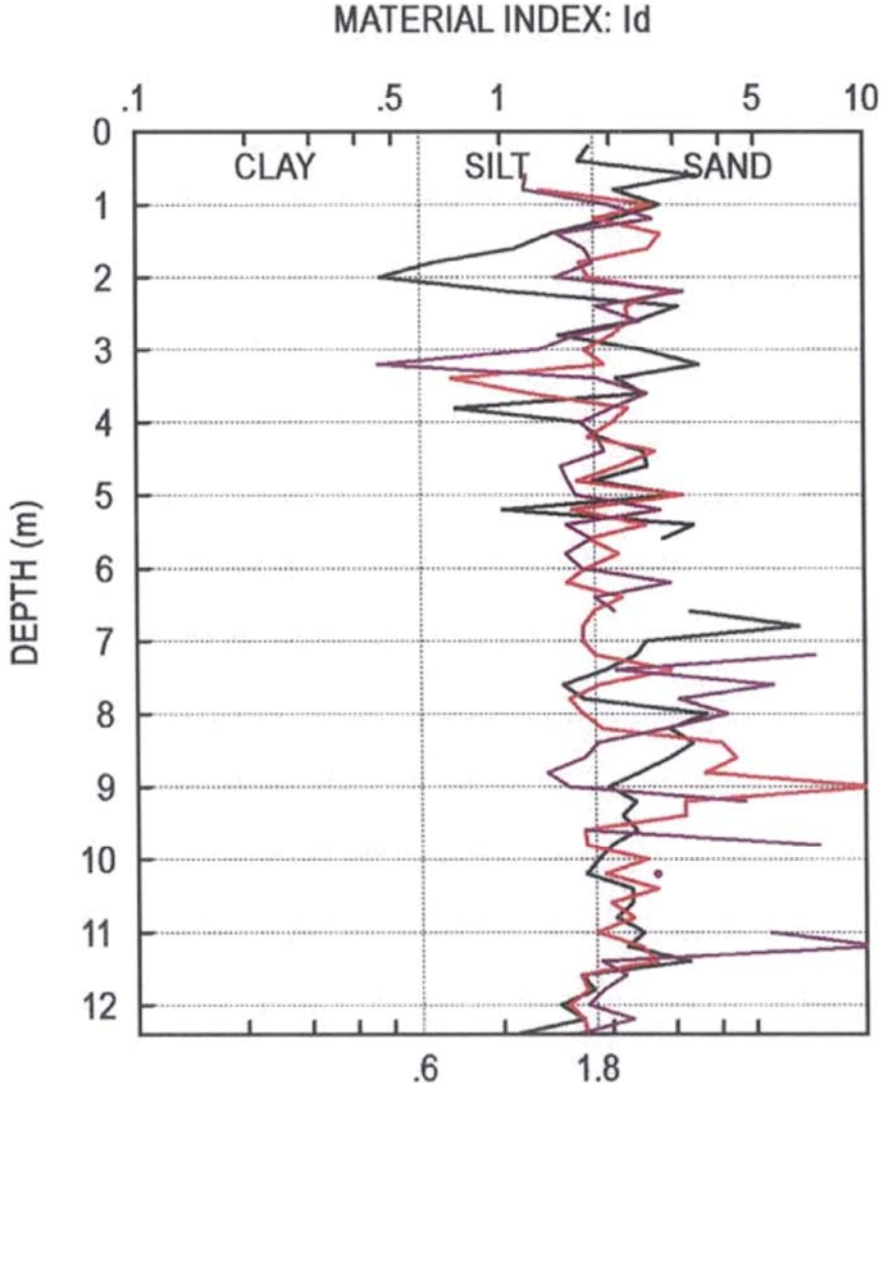
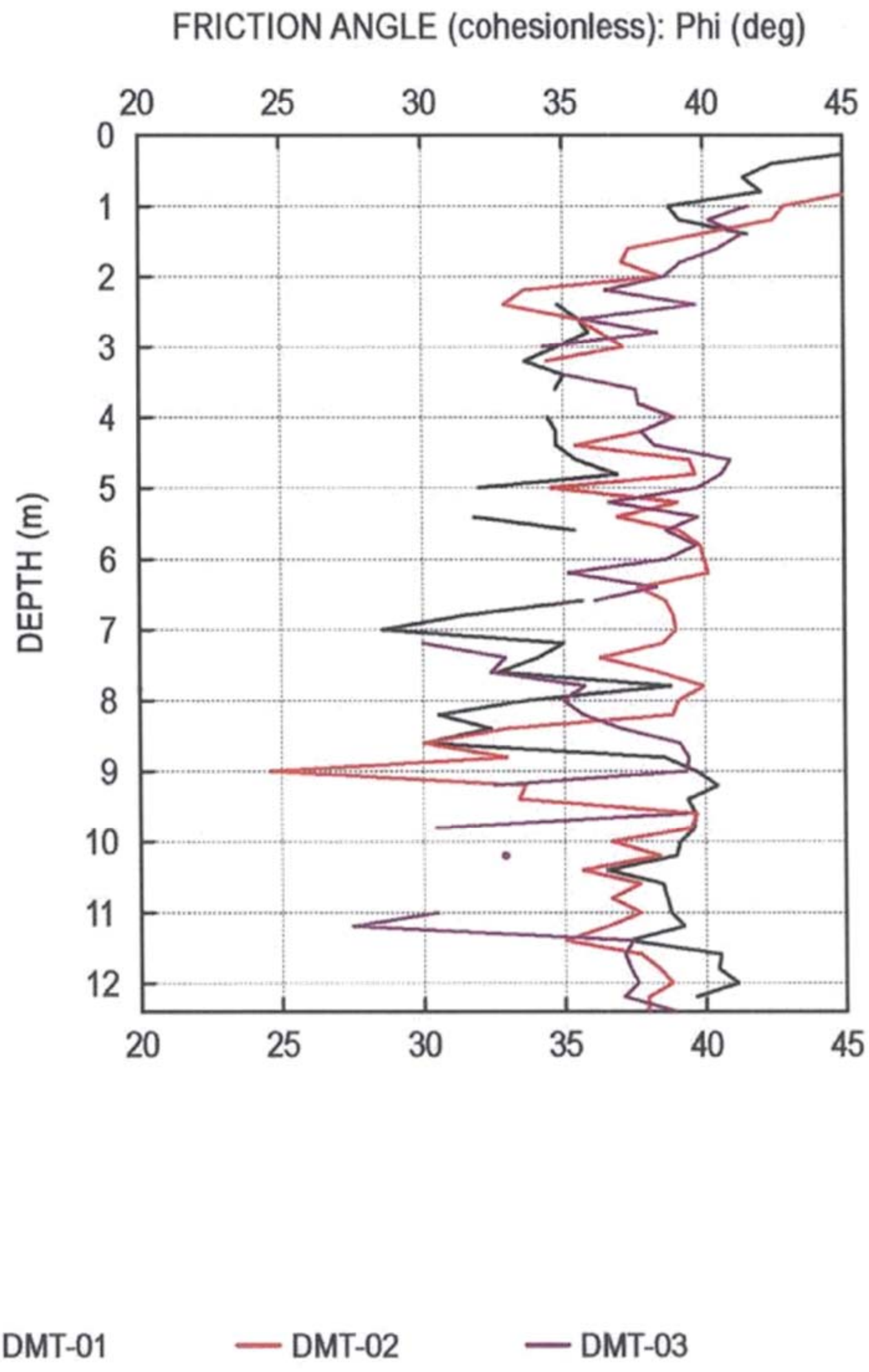
Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd

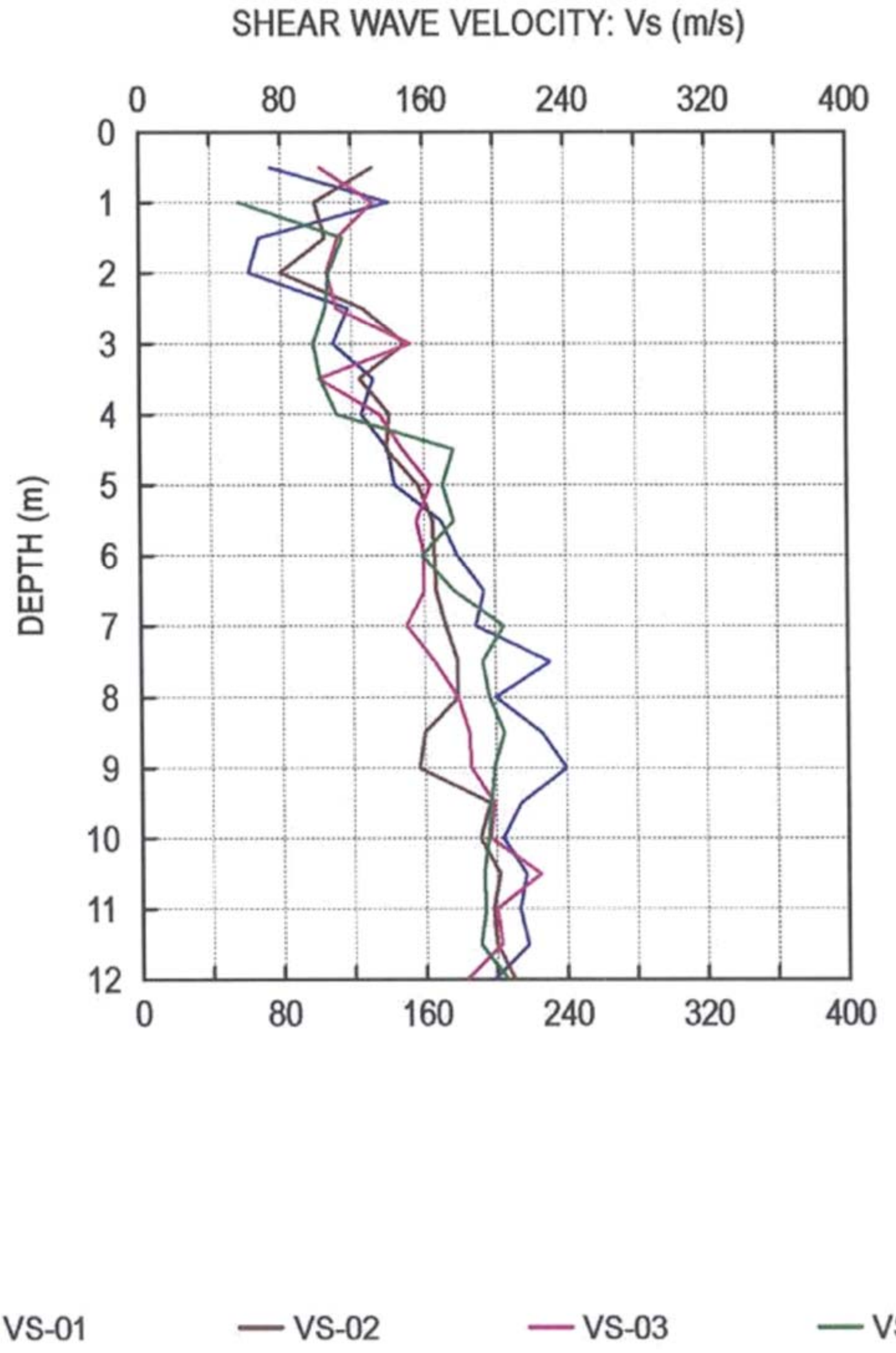
SUPERIMPOSED TEST RESULTS



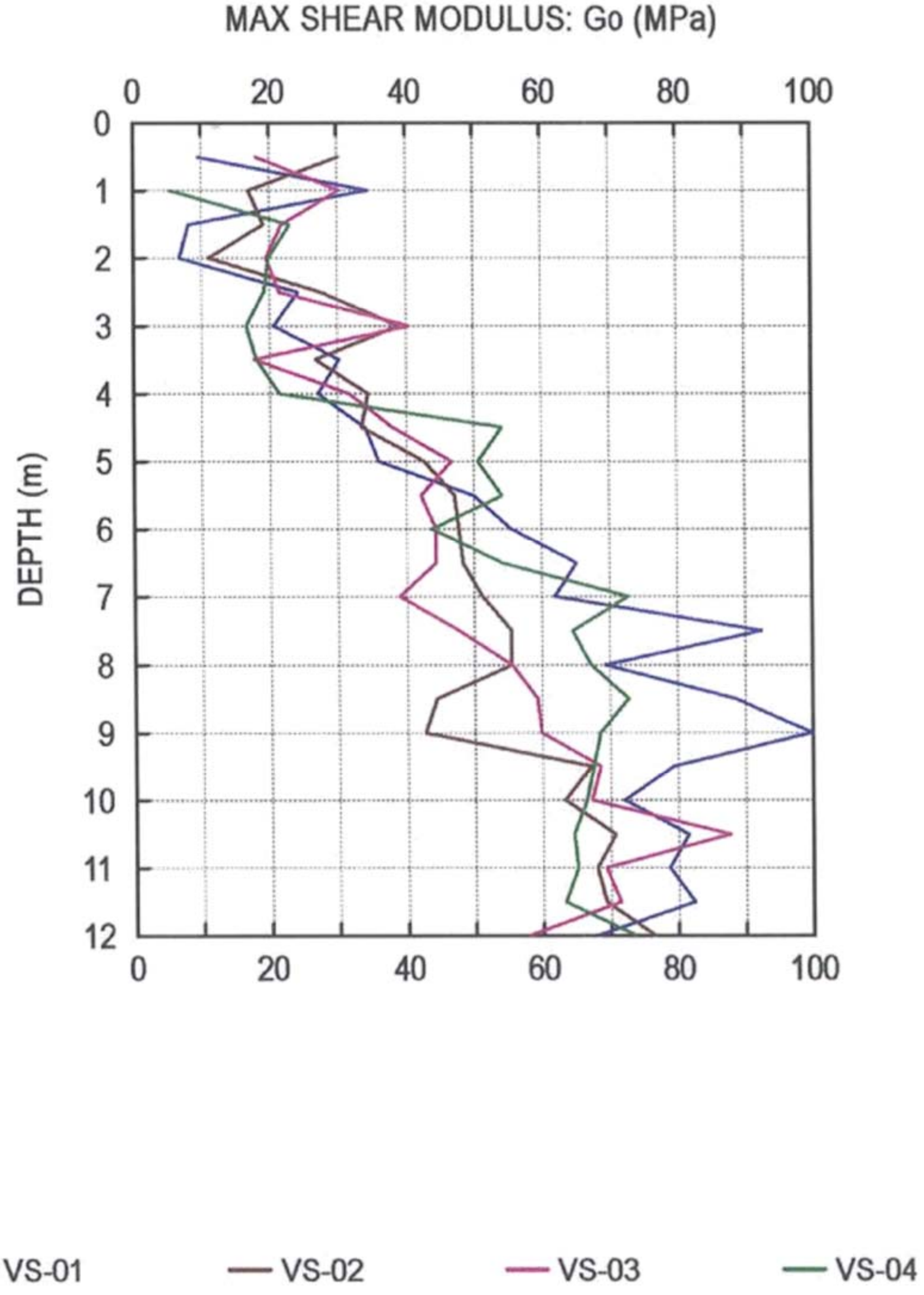
Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd
SUPERIMPOSED TEST RESULTS	



Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd
SUPERIMPOSED TEST RESULTS	



Ground Investigation	Hiways Geotechnical
15-169	27 Shirley Rd
SUPERIMPOSED TEST RESULTS	



DMT-01	LEGEND	INTERPRETED PARAMETERS	GENERAL PARAMETERS
10 SEP 2015 Ground Investigation Hiways Geotechnical 15-169 27 Shirley Rd	Z = Depth Below Ground Level Po,P1,P2 = Corrected A,B,C readings Id = Material Index Ed = Dilatometer Modulus Ud = Pore Press. Index = (P2-Uo)/(Po-Uo) Gamma = Bulk unit weight Sigma' = Effective overb. stress Uo = Pore pressure	Phi = Safe floor value of Friction Angle Ko = In situ earth press. coeff. M = Constrained modulus (at Sigma') Cu = Undrained shear strength Ocr = Overconsolidation ratio (OCR = 'relative OCR'- generally realistic. If accurate independent OCR available, apply suitable factor)	DeltaA = 23 kPa DeltaB = 32 kPa GammaTop = 17.0 kN/m^3 FactorEd = 34.7 ZMCal = 0.0 kPa ZMAB = 0.0 kPa ZMC = 0.0 kPa Zabs = 0.0 m Zw = 3.0 m

WaterTable at 3.00 m
Reduction formulae according to Marchetti, ASCE Geot.Jnl.Mar. 1980, Vol.109, 299-321; Phi according to TC16 ISSMGE, 2001

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	DMT-01 DESCRIPTION
0.2	143	454		153	422		16.7	3	0	1.75	45.1	9.3				46	36.6		SANDY SILT
0.4	91	311		106	279		15.7	7	0	1.64	15.7	6.0				42	17.6		SANDY SILT
0.6	119	577		122	545		17.7	10	0	3.47	12.3	14.7				41	39.6		SAND
0.8	189	623		193	591		17.7	13	0	2.06	14.4	13.8				42	39.3		SILTY SAND
1.0	109	470		117	438		17.7	17	0	2.75	6.9	11.1				39	24.2		SILTY SAND
1.2	145	497		153	465		17.7	20	0	2.04	7.5	10.8				39	24.1		SILTY SAND
1.4	304	763		307	731		17.7	24	0	1.38	12.8	14.7				42	40.2		SANDY SILT
1.6	118	313		134	281		15.7	28	0	1.10	4.9	5.1		1.1	4.0		9.1	18	SILT
1.8	111	247		130	215		15.7	31	0	0.65	4.2	3.0		1.0	3.2		4.8	17	CLAYEY SILT
2.0	104	214	31	124	182	54	15.7	34	0	0.46	3.7	2.0	0.43	0.92	2.6		3.0	16	SILTY CLAY
2.2	125	319		141	287		15.7	37	0	1.03	3.8	5.1		0.95	2.7		7.8	18	SILT
2.4	124	556		128	524		17.7	40	0	3.09	3.2	13.7				35	20.7		SILTY SAND
2.6	155	569		160	537		17.7	44	0	2.36	3.7	13.1				36	20.8		SILTY SAND
2.8	174	482		184	450		16.7	47	0	1.44	3.9	9.2				36	14.6		SANDY SILT
3.0	156	585		160	553		17.7	50	0	2.45	3.2	13.6				35	19.9		SILTY SAND
3.2	136	642		136	610		17.7	52	2	3.52	2.6	16.4				34	21.8		SAND
3.4	176	581		182	549		17.7	54	4	2.07	3.3	12.8				35	18.8		SILTY SAND
3.6	177	643		179	611		17.7	55	6	2.49	3.1	15.0				35	21.8		SILTY SAND
3.8	202	406		218	374		16.7	57	8	0.75	3.7	5.4		0.93	2.6		8.1	27	CLAYEY SILT
4.0	174	502		183	470		16.7	58	10	1.65	3.0	9.9				34	13.4		SANDY SILT
4.2	193	580		199	548		17.7	60	12	1.86	3.2	12.1				35	17.1		SILTY SAND
4.4	206	713		206	681		17.7	61	14	2.46	3.2	16.5				35	24.0		SILTY SAND
4.6	246	844		242	812		18.6	63	16	2.52	3.6	19.8				35	31.3		SILTY SAND
4.8	328	904		325	872		17.7	64	18	1.78	4.8	19.0				37	34.1		SANDY SILT
5.0	144	581		148	549		17.7	66	20	3.13	1.9	13.9				32	15.0		SILTY SAND
5.2	121	288		138	256		15.7	68	22	1.01	1.7	4.1		0.47	<0.8		3.5	12	SILT
5.4	151	625		153	593		17.7	69	24	3.40	1.9	15.3				32	16.0		SAND
5.6	286	1004		276	972		18.6	70	26	2.78	3.6	24.2				35	38.4		SILTY SAND
6.6	355	1338		332	1306		18.6	79	35	3.29	3.7	33.8				36	55.7		SILTY SAND
6.8	203	1158		181	1126		17.7	81	37	6.58	1.8	32.8				32	32.8		SAND
7.0	117	386		129	354		16.7	82	39	2.50	1.1	7.8				29	6.6		SILTY SAND
7.2	326	998		318	966		18.6	84	41	2.34	3.3	22.5				35	33.5		SILTY SAND
7.4	280	771		281	739		18.6	86	43	1.92	2.8	15.9				34	20.6		SILTY SAND
7.6	226	547		236	515		16.7	87	45	1.47	2.2	9.7				33	9.9		SANDY SILT
7.8	688	1723		662	1691		19.1	89	47	1.67	6.9	35.7				39	76.8		SANDY SILT
8.0	304	1184		286	1152		18.6	91	49	3.66	2.6	30.1				34	40.1		SAND
8.2	187	627		191	595		17.7	92	51	2.89	1.5	14.0				31	11.9		SILTY SAND
8.4	256	933		248	901		18.6	94	53	3.35	2.1	22.7				32	25.7		SAND
8.6	184	606		189	574		17.7	96	55	2.88	1.4	13.4				30	11.4		SILTY SAND
8.8	751	2265		701	2233		19.6	97	57	2.38	6.6	53.2				39	113.0		SILTY SAND
9.0	939	2538		885	2506		19.6	99	59	1.96	8.3	56.3				40	131.0		SILTY SAND
9.2	1148	3412		1061	3380		21.1	101	61	2.32	9.9	80.5				40	200.3		SILTY SAND

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	DMT-01 DESCRIPTION
9.4	932	2637		872	2605		19.6	103	63	2.14	7.8	60.1				39	136.7		SILTY SAND
9.6	1012	3014		938	2982		21.1	105	65	2.34	8.3	70.9				40	165.1		SILTY SAND
9.8	1007	2728		947	2696		21.1	108	67	1.99	8.2	60.7				40	140.3		SILTY SAND
10.0	931	2407	54	883	2375	77	19.6	110	69	1.83	7.4	51.8	0.01			39	114.8		SILTY SAND
10.2	913	2264		871	2232		19.1	112	71	1.70	7.2	47.2				39	103.0		SANDY SILT
10.4	603	1735		572	1703		19.6	114	73	2.26	4.4	39.2				37	68.5		SILTY SAND
10.6	884	2564		826	2532		19.6	116	75	2.27	6.5	59.2				38	124.6		SILTY SAND
10.8	915	2503		861	2471		19.6	118	77	2.05	6.7	55.9				39	118.6		SILTY SAND
11.0	979	2962	57	906	2930	80	21.1	120	78	2.45	6.9	70.2	0.00			39	152.2		SILTY SAND
11.2	1075	3047		1002	3015		21.1	122	80	2.18	7.6	69.8				39	156.6		SILTY SAND
11.4	788	2798		713	2766		19.6	124	82	3.25	5.1	71.2				37	136.2		SILTY SAND
11.6	1432	3458		1356	3426		20.6	126	84	1.63	10.1	71.8				41	180.1		SANDY SILT
11.8	1440	3633		1356	3601		20.6	128	86	1.77	9.9	77.9				40	194.0		SANDY SILT
12.0	1672	3776	86	1593	3744	109	20.6	130	88	1.43	11.5	74.7	0.01			41	196.7		SANDY SILT
12.2	1251	3044		1187	3012		20.6	133	90	1.66	8.3	63.3				40	146.8		SANDY SILT
12.4	1045	2065		1020	2033		19.1	135	92	1.09	6.9	35.2		1.4	6.9		74.9	139	SILT

DMT-02	LEGEND	INTERPRETED PARAMETERS	GENERAL PARAMETERS
11 SEP 2015 Ground Investigation Hiways Geotechnical 15-169 27 Shirley Rd 1m south from CPT	Z = Depth Below Ground Level Po,P1,P2 = Corrected A,B,C readings Id = Material Index Ed = Dilatometer Modulus Ud = Pore Press. Index = (P2-Uo)/(Po-Uo) Gamma = Bulk unit weight Sigma' = Effective overb. stress Uo = Pore pressure	Phi = Safe floor value of Friction Angle Ko = In situ earth press. coeff. M = Constrained modulus (at Sigma') Cu = Undrained shear strength Ocr = Overconsolidation ratio (OCR = 'relative OCR'- generally realistic. If accurate independent OCR available, apply suitable factor)	DeltaA = 16 kPa DeltaB = 65 kPa GammaTop = 17.0 kN/m^3 FactorEd = 34.7 ZMCal = 0.0 kPa ZMAB = 0.0 kPa ZMC = 0.0 kPa Zabs = 0.0 m Zw = 3.0 m

WaterTable at 3.00 m
Reduction formulae according to Marchetti, ASCE Geot.Jnl.Mar. 1980, Vol.109, 299-321; Phi according to TC16 ISSMGE, 2001

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	DMT-02 DESCRIPTION
0.8	486	1144		473	1079		17.7	14	0	1.28	34.8	21.0				46	77.4		SANDY SILT
1.0	319	1140		298	1075		18.6	17	0	2.61	17.4	27.0				43	81.5		SILTY SAND
1.2	344	1000		331	935		18.6	21	0	1.82	15.9	21.0				43	61.5		SILTY SAND
1.4	228	877		216	812		18.6	25	0	2.77	8.8	20.7				40	49.3		SILTY SAND
1.6	148	585		146	520		17.7	28	0	2.56	5.2	13.0				37	24.7		SILTY SAND
1.8	154	483		158	418		16.7	32	0	1.65	4.9	9.0				37	16.5		SANDY SILT
2.0	232	694		229	629		16.7	35	0	1.75	6.5	13.9				38	29.0		SANDY SILT
2.2	97	450		99	385		16.7	39	0	2.87	2.6	9.9				34	13.0		SILTY SAND
2.4	89	370		95	305		16.7	42	0	2.21	2.3	7.3				33	8.3		SILTY SAND
2.6	166	599		164	534		17.7	45	0	2.25	3.6	12.8				36	20.2		SILTY SAND
2.8	212	695		208	630		17.7	49	0	2.03	4.3	14.6				36	25.0		SILTY SAND
3.0	265	765		260	700		17.7	52	0	1.69	5.0	15.3				37	28.0		SANDY SILT
3.2	161	536		162	471		17.7	54	2	1.93	3.0	10.7				34	14.6		SILTY SAND
3.4	147	335		158	270		15.7	55	4	0.73	2.8	3.9		0.73	1.7		4.7	18	CLAYEY SILT
3.6	174	453		180	388		16.7	57	6	1.19	3.1	7.2		0.80	2.0		9.7	21	SILT
3.8	352	1126		333	1061		18.6	58	8	2.24	5.6	25.2				38	49.8		SILTY SAND
4.0	465	1373		440	1308		18.6	60	10	2.02	7.2	30.1				39	66.1		SILTY SAND
4.2	373	1029		360	964		17.7	61	12	1.73	5.7	21.0				38	41.1		SANDY SILT
4.4	252	904		239	839		18.6	63	14	2.66	3.6	20.8				35	33.0		SILTY SAND
4.6	573	1703		537	1638		19.6	65	16	2.11	8.0	38.2				39	87.8		SILTY SAND
4.8	606	1551		579	1486		19.1	67	18	1.62	8.4	31.5				40	73.5		SANDY SILT
5.0	245	956	10	230	891	26	18.6	69	20	3.15	3.1	23.0	0.03			35	33.8		SILTY SAND
5.2	561	1410		539	1345		19.1	70	22	1.56	7.3	28.0				39	61.6		SANDY SILT
5.4	393	1293		368	1228		18.6	72	24	2.50	4.8	29.8				37	54.7		SILTY SAND
5.6	602	1587		573	1522		19.1	74	26	1.73	7.4	32.9				39	72.9		SANDY SILT
5.8	734	2133		684	2068		19.6	76	27	2.11	8.7	48.0				40	113.6		SILTY SAND
6.0	768	1981	20	727	1916	36	19.1	78	29	1.70	9.0	41.2	0.01			40	98.8		SANDY SILT
6.2	805	1944		768	1879		19.1	80	31	1.51	9.2	38.5				40	93.4		SANDY SILT
6.4	510	1509		480	1444		19.6	82	33	2.16	5.5	33.4				38	65.0		SILTY SAND
6.6	624	1654		593	1589		19.1	84	35	1.79	6.7	34.6				39	73.2		SANDY SILT
6.8	675	1723		643	1658		19.1	85	37	1.68	7.1	35.2				39	76.5		SANDY SILT
7.0	701	1778	29	667	1713	45	19.1	87	39	1.67	7.2	36.3	0.01			39	79.3		SANDY SILT
7.2	662	1751		628	1686		19.6	89	41	1.80	6.6	36.7				39	77.3		SILTY SAND
7.4	465	1617		427	1552		18.6	91	43	2.93	4.2	39.0				36	68.1		SILTY SAND
7.6	671	1794		635	1729		19.6	93	45	1.86	6.4	38.0				38	78.7		SILTY SAND
7.8	937	2249		891	2184		19.1	95	47	1.53	8.9	44.9				40	107.1		SANDY SILT
8.0	800	2011	37	760	1946	53	19.1	97	49	1.67	7.3	41.2	0.01			39	90.8		SANDY SILT
8.2	788	2108		742	2043		19.6	99	51	1.88	7.0	45.1				39	97.8		SILTY SAND
8.4	313	1283		285	1218		18.6	101	53	4.03	2.3	32.4				33	39.7		SAND
8.6	210	885		196	820		17.7	102	55	4.41	1.4	21.6				30	18.4		SAND
8.8	320	1217		295	1152		18.6	104	57	3.60	2.3	29.7				33	36.3		SAND
9.0	137	846	41	122	781	57	17.7	106	59	10.51	0.6	22.9	-0.03			25	19.4		SAND

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	DMT-02 DESCRIPTION
9.2	362	1276		336	1211		18.6	107	61	3.17	2.6	30.4				34	40.1		SILTY SAND
9.4	357	1258		332	1193		18.6	109	63	3.20	2.5	29.9				33	38.4		SILTY SAND
9.6	1055	2621		997	2556		20.6	111	65	1.67	8.4	54.1				40	126.4		SANDY SILT
9.8	1032	2587		974	2522		20.6	113	67	1.71	8.0	53.7				39	123.1		SANDY SILT
10.0	639	1968	51	593	1903	67	19.6	115	69	2.50	4.6	45.5	0.00			37	81.5		SILTY SAND
10.2	866	2304		814	2239		19.6	117	71	1.92	6.4	49.4				38	102.6		SILTY SAND
10.4	555	1762		515	1697		19.6	119	73	2.67	3.7	41.0				36	66.4		SILTY SAND
10.6	789	2129		742	2064		19.6	121	75	1.98	5.5	45.9				38	89.2		SILTY SAND
10.8	677	1971		632	1906		19.6	123	77	2.29	4.5	44.2				37	78.4		SILTY SAND
11.0	812	2083	59	768	2018	75	19.6	125	78	1.81	5.5	43.4	-0.01			38	84.1		SILTY SAND
11.2	688	2041		640	1976		19.6	127	80	2.39	4.4	46.3				37	81.4		SILTY SAND
11.4	545	1698		507	1633		19.6	129	82	2.65	3.3	39.1				35	59.0		SILTY SAND
11.6	849	2057		809	1992		19.1	131	84	1.63	5.5	41.1				38	79.4		SANDY SILT
11.8	974	2409		922	2344		19.1	133	86	1.70	6.3	49.3				38	101.6		SANDY SILT
12.0	1075	2504	75	1024	2439	91	20.6	134	88	1.51	7.0	49.1	0.00			39	105.6		SANDY SILT
12.2	936	2281		889	2216		19.1	137	90	1.66	5.8	46.1				38	91.5		SANDY SILT
12.4	945	2324		896	2259		19.1	138	92	1.70	5.8	47.3				38	93.7		SANDY SILT

DMT-03	LEGEND	INTERPRETED PARAMETERS	GENERAL PARAMETERS
10 SEP 2015	Z = Depth Below Ground Level	Phi = Safe floor value of Friction Angle	DeltaA = 11 kPa
Ground Investigation	Po,P1,P2 = Corrected A,B,C readings	Ko = In situ earth press. coeff.	DeltaB = 72 kPa
Hiways Geotechnical	Id = Material Index	M = Constrained modulus (at Sigma')	GammaTop = 17.0 kN/m^3
15-169	Ed = Dilatometer Modulus	Cu = Undrained shear strength	FactorEd = 34.7
27 Shirley Rd	Ud = Pore Press. Index = (P2-Uo)/(Po-Uo)	Ocr = Overconsolidation ratio	ZMCal = 0.0 kPa
	Gamma = Bulk unit weight	(OCR = 'relative OCR'- generally	ZMAB = 0.0 kPa
	Sigma' = Effective overb. stress	realistic. If accurate independent OCR	ZMC = 0.0 kPa
	Uo = Pore pressure	available, apply suitable factor)	Zabs = 0.0 m
			Zw = 3.0 m

WaterTable at 3.00 m
Reduction formulae according to Marchetti, ASCE Geot.Jnl.Mar. 1980, Vol.109, 299-321; Phi according to TC16 ISSMGE, 2001

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	DMT-03 DESCRIPTION
0.6	253	618		250	546		16.7	10	0	1.18	24.5	10.3		3.1	50.0		34.4	51	SILT
0.8	239	584		237	512		16.7	14	0	1.16	17.5	9.5		2.6	29.6		28.9	45	SILT
1.0	227	723		217	651		17.7	17	0	2.00	12.9	15.0				42	41.2		SILTY SAND
1.2	204	764		191	692		17.7	20	0	2.62	9.4	17.4				40	42.4		SILTY SAND
1.4	300	779		291	707		17.7	24	0	1.43	12.2	14.4				41	38.8		SANDY SILT
1.6	288	820		277	748		17.7	27	0	1.70	10.1	16.4				41	41.0		SANDY SILT
1.8	244	729		235	657		16.7	31	0	1.80	7.6	14.6				39	32.8		SANDY SILT
2.0	234	627		230	555		16.7	34	0	1.42	6.7	11.3				39	23.8		SANDY SILT
2.2	181	771		167	699		17.7	38	0	3.19	4.4	18.5				37	33.1		SILTY SAND
2.4	368	1064		348	992		18.6	41	0	1.85	8.5	22.3				40	52.3		SILTY SAND
2.6	177	648		169	576		17.7	45	0	2.42	3.8	14.1				36	22.8		SILTY SAND
2.8	320	875		307	803		17.7	48	0	1.61	6.3	17.2				38	35.5		SANDY SILT
3.0	149	416		151	344		16.7	52	0	1.28	2.9	6.7				34	8.6		SANDY SILT
3.2	162	318	74	169	246	85	15.7	53	2	0.46	3.1	2.7	0.50	0.81	2.0		3.5	21	SILTY CLAY
3.4	192	596		187	524		17.7	55	4	1.84	3.4	11.7				35	17.2		SILTY SAND
3.6	336	1153		310	1081		18.6	56	6	2.53	5.4	26.7				38	52.2		SILTY SAND
3.8	348	1047		328	975		18.6	58	8	2.02	5.5	22.4				38	43.7		SILTY SAND
4.0	456	1213		433	1141		17.7	60	10	1.67	7.1	24.6				39	53.4		SANDY SILT
4.2	376	1061	7	357	989	18	18.6	61	12	1.83	5.6	21.9	0.02			38	43.0		SILTY SAND
4.4	431	1233		406	1161		18.6	63	14	1.92	6.2	26.2				38	53.9		SILTY SAND
4.6	774	1856		735	1784		19.1	65	16	1.46	11.1	36.4				41	94.6		SANDY SILT
4.8	741	1817		702	1745		19.1	67	18	1.52	10.3	36.2				41	91.4		SANDY SILT
5.0	635	1606		602	1534		19.1	68	20	1.60	8.5	32.4				40	75.8		SANDY SILT
5.2	368	1277	13	338	1205	24	18.6	70	22	2.74	4.5	30.1	0.01			37	53.9		SILTY SAND
5.4	673	1644		640	1572		19.1	72	24	1.51	8.5	32.4				40	76.0		SANDY SILT
5.6	553	1468		522	1396		19.1	74	26	1.76	6.7	30.3				39	64.3		SANDY SILT
5.8	704	1703		669	1631		19.1	76	27	1.50	8.5	33.4				40	78.1		SANDY SILT
6.0	581	1490		551	1418		19.1	78	29	1.66	6.7	30.1				39	63.8		SANDY SILT
6.2	329	1167	22	302	1095	33	18.6	80	31	2.93	3.4	27.5	0.01			35	42.9		SILTY SAND
6.4	576	1536		543	1464		19.6	81	33	1.81	6.3	32.0				38	65.8		SILTY SAND
6.6	397	1142		375	1070		18.6	83	35	2.05	4.1	24.1				36	40.2		SILTY SAND
7.2	195	1130	28	163	1058	39	17.7	89	41	7.32	1.4	31.0	-0.02			30	26.4		SAND
7.4	258	745		249	673		17.7	90	43	2.06	2.3	14.7				33	16.6		SILTY SAND
7.6	275	1376		235	1304		18.6	92	45	5.63	2.1	37.1				32	42.0		SAND
7.8	444	1567		403	1495		18.6	93	47	3.07	3.8	37.9				36	62.9		SILTY SAND
8.0	413	1741		362	1669		18.6	95	49	4.18	3.3	45.4				35	69.5		SAND
8.2	454	1512	36	416	1440	47	18.6	97	51	2.80	3.8	35.5	-0.01			36	58.2		SILTY SAND
8.4	562	1479		531	1407		19.6	99	53	1.83	4.8	30.4				37	55.1		SILTY SAND
8.6	850	2131		801	2059		19.1	101	55	1.69	7.4	43.6				39	96.6		SANDY SILT
8.8	911	2022		871	1950		19.1	103	57	1.33	7.9	37.5				39	85.1		SANDY SILT
9.0	917	2177		869	2105		19.1	104	59	1.53	7.8	42.9				39	96.7		SANDY SILT
9.2	324	1407	43	285	1335	54	18.6	106	61	4.68	2.1	36.4	-0.03			33	41.8		SAND

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	DMT-03 DESCRIPTION
9.4	646	731		657	659		13.7	108	63		5.5	0.1		1.2	4.9		0.1	84	MUD AND/OR PEAT
9.6	960	2395		903	2323		19.1	109	65	1.69	7.7	49.3				39	110.9		SANDY SILT
9.8	280	1544		232	1472		18.6	111	67	7.50	1.5	43.0				30	36.6		SAND
10.2	351	1088		329	1016		18.6	114	71	2.65	2.3	23.8				33	28.0		SILTY SAND
11.0	297	1327		261	1255		18.6	121	78	5.46	1.5	34.5				31	29.4		SAND
11.2	241	1486		194	1414		17.7	123	80	10.75	0.9	42.3				27	36.0		SAND
11.4	783	2027		736	1955		19.6	125	82	1.87	5.2	42.3				37	80.0		SILTY SAND
11.6	762	2141		708	2069		19.6	127	84	2.18	4.9	47.2				37	87.2		SILTY SAND
11.8	797	2063		749	1991		19.6	129	86	1.87	5.2	43.1				37	80.8		SILTY SAND
12.0	844	2083		797	2011		19.1	131	88	1.71	5.4	42.1				38	80.8		SANDY SILT
12.2	801	2292	68	742	2220	79	19.6	132	90	2.27	4.9	51.3	-0.02			37	94.9		SILTY SAND
12.4	1099	2637		1037	2565		20.6	134	92	1.62	7.0	53.0				39	114.6		SANDY SILT

VS-01 - Tabular data: Vs, Go, Vs Repeatability

Each Vs value in the 'Vs Repeatability' column corresponds to a distinct energization.

Z	Vs	Go	Rho	Vs Repeatability	Var Coeff.
[m]	[m/s]	[MPa]	[kg/m^3]	[m/s]	[%]
0.50	74	9.5	1733	75,74,74	0.78
1.00	141	34.5	1733	141,142,139	0.92
1.50	68	8.0	1733	68,67,69	1.20
2.00	62	6.7	1733	62,63,62	0.93
2.50	118	24.1	1733	115,119,120	1.83
3.00	109	20.6	1733	111,106,110	1.98
3.50	132	30.2	1733	131,133	0.76
4.00	125	27.1	1733	123,125,126	1.03
4.50	140	34.0	1733	133,143,143	3.38
5.00	144	35.9	1733	143,144,145	0.57
5.50	170	50.1	1733	168,170,171	0.76
6.00	179	55.5	1733	177,179,181	0.91
6.50	194	65.2	1733	194,194	0.00
7.00	189	61.9	1733	189,190,188	0.43
7.50	231	92.5	1733	233,231,229	0.71
8.00	200	69.3	1733	200,200,199	0.29
8.50	226	88.5	1733	227,225,225	0.44
9.00	240	99.8	1733	240,240,239	0.24
9.50	214	79.4	1733	215,213,213	0.47
10.00	204	72.1	1733	199,206,207	1.74
10.50	217	81.6	1733	213,219,220	1.43
11.00	213	78.6	1733	213,215,211	0.77
11.50	218	82.4	1733	215,220,220	1.09
12.00	198	67.9	1733	198,196,201	1.05

VS-02 - Tabular data: Vs, Go, Vs Repeatability

Each Vs value in the 'Vs Repeatability' column corresponds to a distinct energization.

Z	Vs	Go	Rho	Vs Repeatability	Var Coeff.
[m]	[m/s]	[MPa]	[kg/m^3]	[m/s]	[%]
0.50	132	30.2	1733	134,131	1.20
1.00	99	17.0	1733	103,97,97	2.86
1.50	105	19.1	1733	98,107,109	4.57
2.00	80	11.1	1733	77,80,82	2.60
2.50	126	27.5	1733	130,127,121	2.97
3.00	150	39.0	1733	149,150,150	0.38
3.50	124	26.6	1733	124,123,126	1.04
4.00	141	34.5	1733	139,142,143	1.23
4.50	139	33.5	1733	139,139,140	0.42
5.00	157	42.7	1733	154,158,158	1.22
5.50	165	47.2	1733	165,167,164	0.78
6.00	166	47.8	1733	165,166,167	0.49
6.50	167	48.3	1733	167,166,167	0.35
7.00	172	51.3	1733	173,171,173	0.58
7.50	179	55.5	1733	178,179,181	0.72
8.00	179	55.5	1733	178,177,181	0.97
8.50	160	44.4	1733	161,159,161	0.63
9.00	157	42.7	1733	157,160,155	1.33
9.50	197	67.3	1733	195,197,198	0.66
10.00	191	63.2	1733	190,194,188	1.32
10.50	202	70.7	1733	201,203,202	0.40
11.00	198	67.9	1733	199,198,197	0.41
11.50	200	69.3	1733	201,201,199	0.50
12.00	210	76.4	1733	205,212,214	1.84

VS-03 - Tabular data: Vs, Go, Vs Repeatability

Each Vs value in the 'Vs Repeatability' column corresponds to a distinct energization.

Z	Vs	Go	Rho	Vs Repeatability	Var Coeff.
[m]	[m/s]	[MPa]	[kg/m^3]	[m/s]	[%]
0.50	102	18.0	1733	102,103	0.69
1.00	132	30.2	1733	132,134,131	0.98
1.50	112	21.7	1733	112,110,114	1.46
2.00	106	19.5	1733	100,107,110	3.97
2.50	111	21.4	1733	107,112,113	2.38
3.00	153	40.6	1733	148,155,155	2.17
3.50	101	17.7	1733	102,102,100	0.99
4.00	135	31.6	1733	132,134,138	1.86
4.50	148	38.0	1733	149,148,148	0.39
5.00	164	46.6	1733	164,162,166	1.00
5.50	156	42.2	1733	156,156,157	0.37
6.00	160	44.4	1733	161,160,158	0.81
6.50	160	44.4	1733	163,158,159	1.35
7.00	150	39.0	1733	150,151,150	0.38
7.50	166	47.8	1733	166,164,167	0.78
8.00	179	55.5	1733	179,180,179	0.32
8.50	185	59.3	1733	185,183,186	0.70
9.00	186	60.0	1733	183,192,183	2.28
9.50	199	68.6	1733	198,198,200	0.50
10.00	197	67.3	1733	196,195,199	0.88
10.50	225	87.7	1733	223,227,224	0.77
11.00	200	69.3	1733	203,197,199	1.26
11.50	203	71.4	1733	203,205,202	0.64
12.00	183	58.0	1733	180,186,182	1.38

VS-04 - Tabular data: Vs, Go, Vs Repeatability

Each Vs value in the 'Vs Repeatability' column corresponds to a distinct energization.

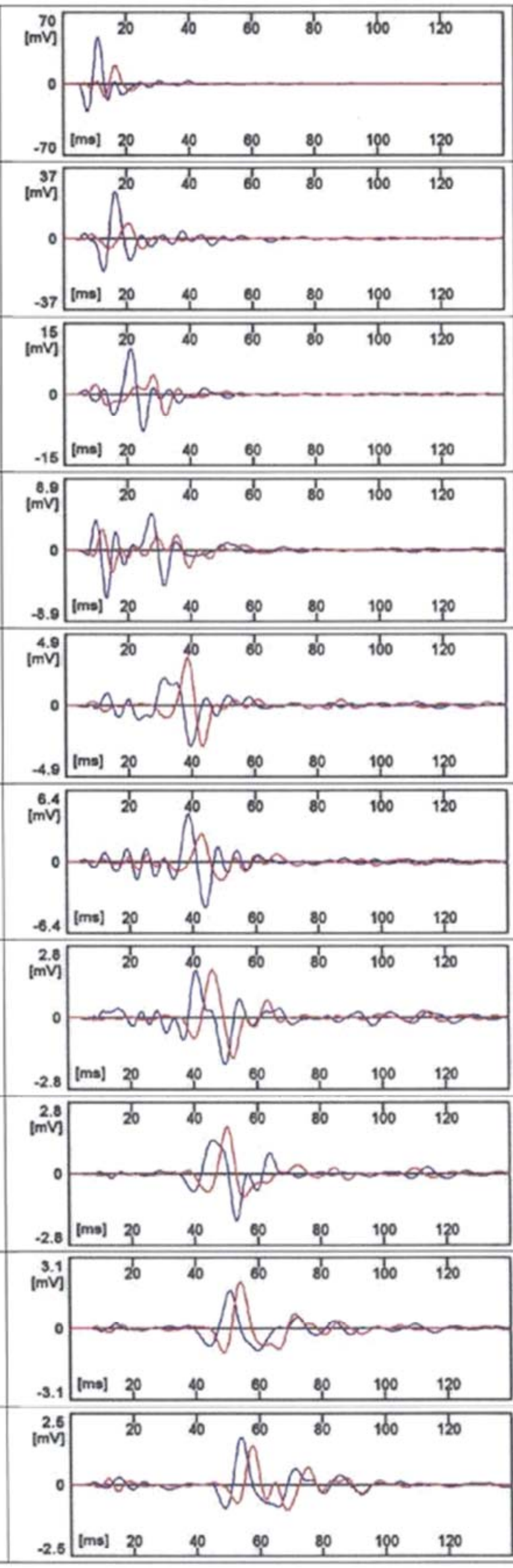
Z	Vs	Go	Rho	Vs Repeatability	Var Coeff.
[m]	[m/s]	[MPa]	[kg/m^3]	[m/s]	[%]
1.00	56	5.4	1733	53,57,57	3.42
1.50	115	22.9	1733	110,122,114	4.35
2.00	107	19.8	1733	106,107,108	0.76
2.50	105	19.1	1733	105,104,107	1.23
3.00	98	16.6	1733	96,99,100	1.77
3.50	102	18.0	1733	100,101,106	2.59
4.00	111	21.4	1733	112,111,110	0.74
4.50	177	54.3	1733	174,175,181	1.76
5.00	171	50.7	1733	172,173,169	1.01
5.50	177	54.3	1733	179,176,176	0.80
6.00	159	43.8	1733	160,158,158	0.63
6.50	177	54.3	1733	176,179,177	0.73
7.00	205	72.8	1733	209,203,202	1.52
7.50	193	64.5	1733	193,193,192	0.30
8.00	197	67.3	1733	197,197,197	0.00
8.50	205	72.8	1733	205,205,205	0.00
9.00	199	68.6	1733	199,199,196,200,200,200	0.71
10.00	196	66.6	1733	196,194,197	0.66
10.50	193	64.5	1733	192,192,194	0.52
11.00	194	65.2	1733	197,192,194	1.07
11.50	191	63.2	1733	193,191,190	0.68
12.00	206	73.5	1733	210,205,202	1.61

VS-01 - Vs

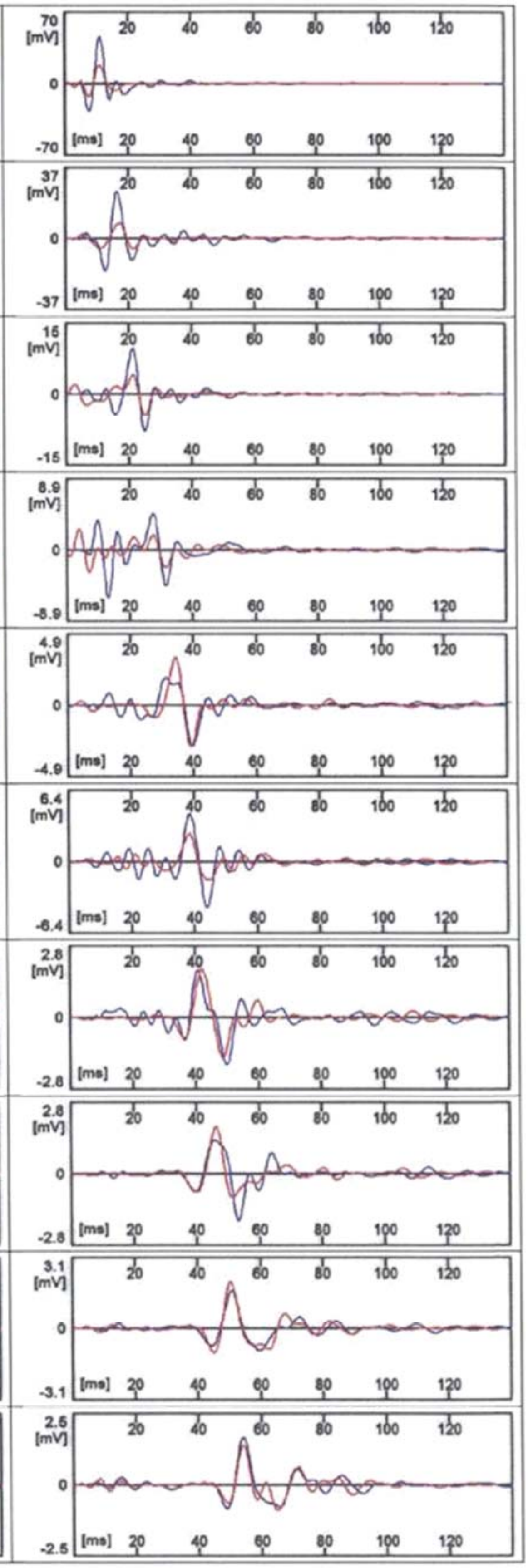
RECORDED / RE-PHASED

Z = 0.50 m	Ds = 0.42 m	Dt = 5.60 ms	Vs = 75 m/s	10 SEP 2015	10:36:19
Z = 1.00 m	Ds = 0.48 m	Dt = 3.40 ms	Vs = 141 m/s	10 SEP 2015	10:37:27
Z = 1.50 m	Ds = 0.49 m	Dt = 7.12 ms	Vs = 69 m/s	10 SEP 2015	10:40:25
Z = 2.00 m	Ds = 0.49 m	Dt = 7.92 ms	Vs = 62 m/s	10 SEP 2015	10:41:04
Z = 2.50 m	Ds = 0.50 m	Dt = 4.33 ms	Vs = 115 m/s	10 SEP 2015	10:43:17
Z = 3.00 m	Ds = 0.50 m	Dt = 4.47 ms	Vs = 111 m/s	10 SEP 2015	10:44:20
Z = 3.50 m	Ds = 0.50 m	Dt = 3.80 ms	Vs = 131 m/s	10 SEP 2015	10:47:51
Z = 4.00 m	Ds = 0.50 m	Dt = 4.06 ms	Vs = 123 m/s	10 SEP 2015	10:48:37
Z = 4.50 m	Ds = 0.50 m	Dt = 3.75 ms	Vs = 133 m/s	10 SEP 2015	10:50:20
Z = 5.00 m	Ds = 0.50 m	Dt = 3.49 ms	Vs = 143 m/s	10 SEP 2015	10:51:43

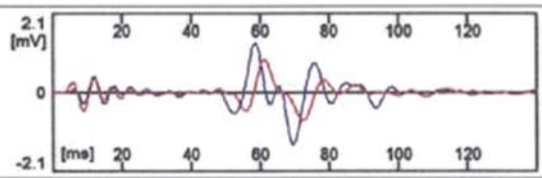
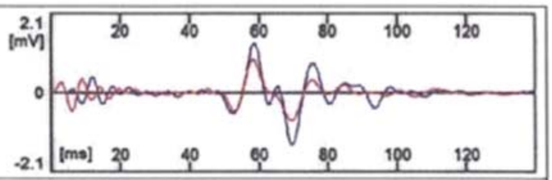
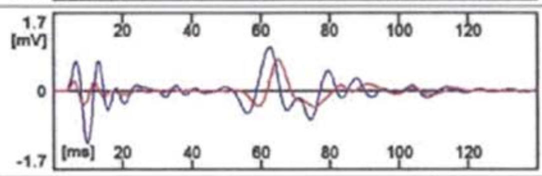
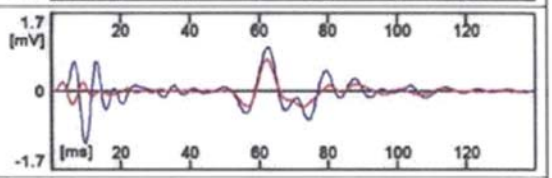
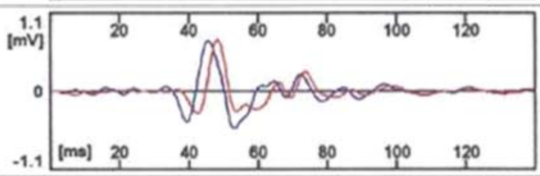
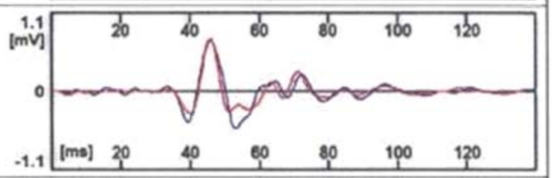
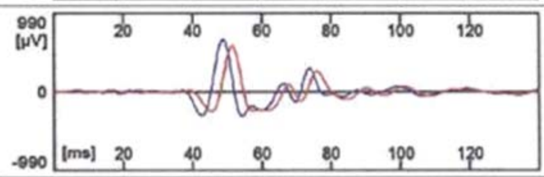
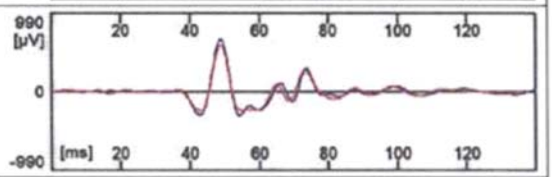
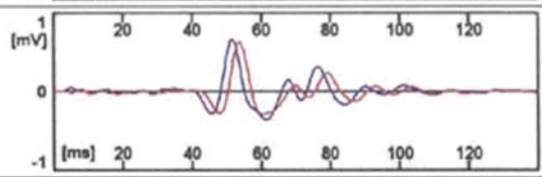
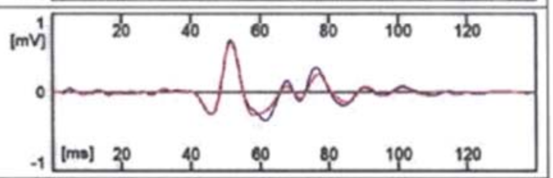
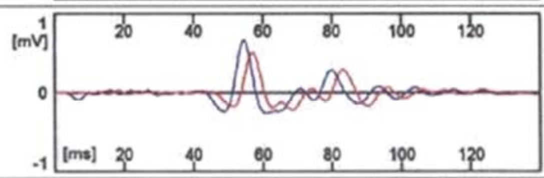
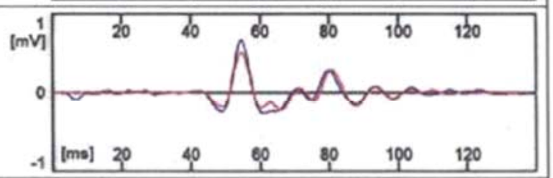
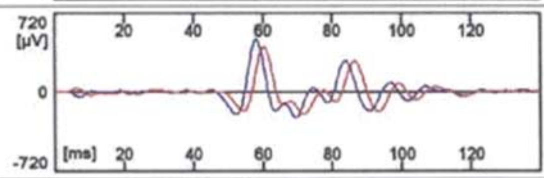
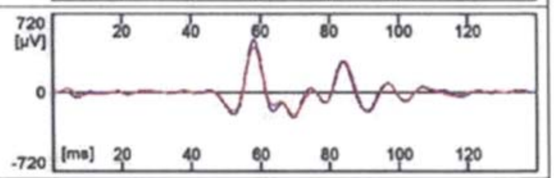
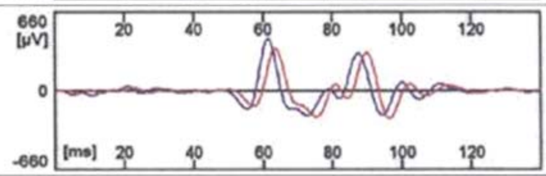
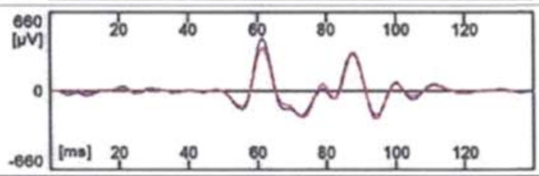
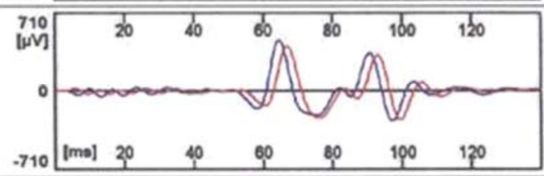
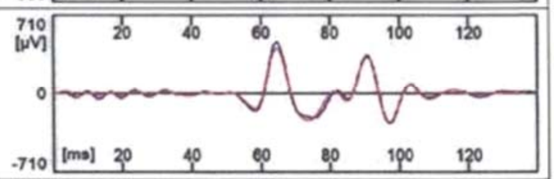
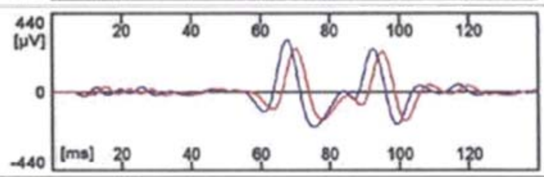
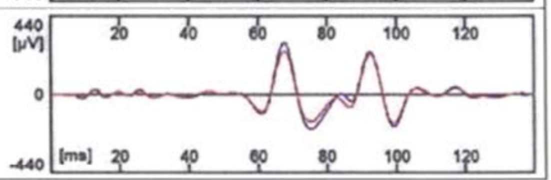
RECORDED



RE-PHASED

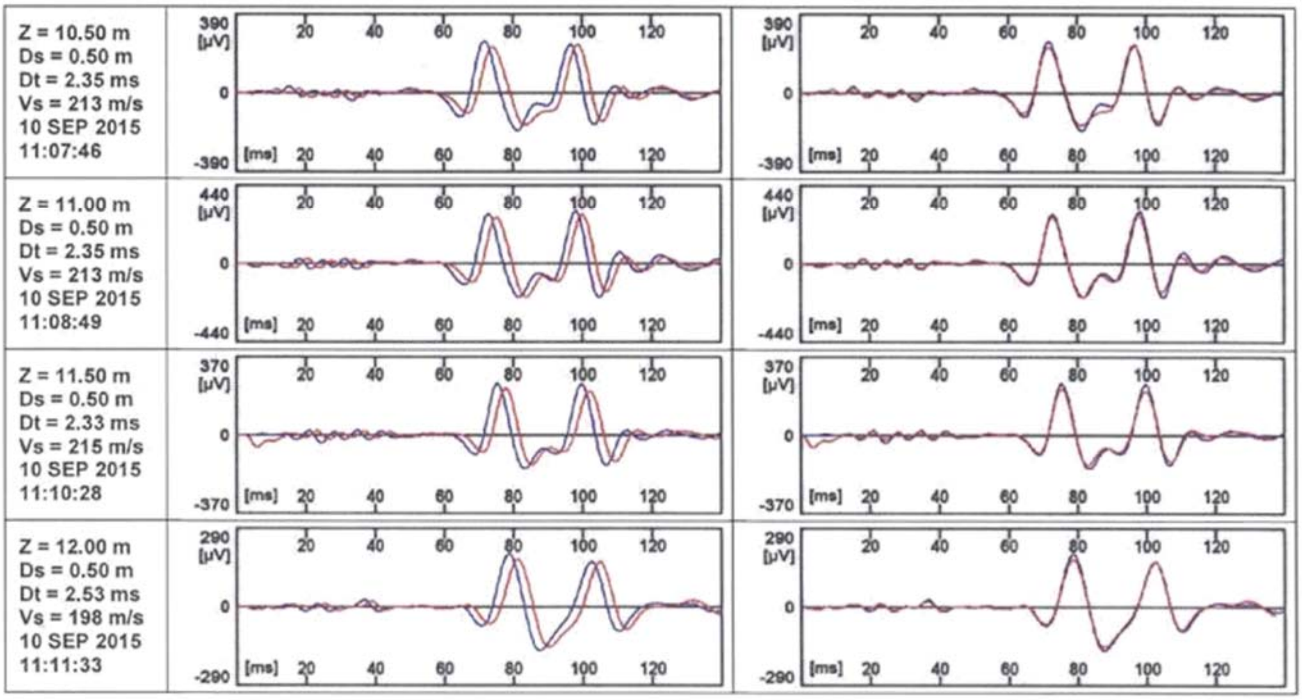


VS-01 - Vs

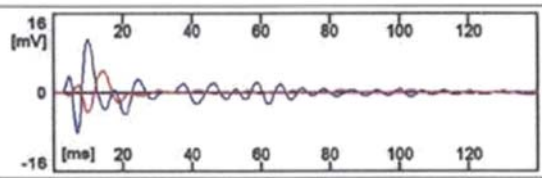
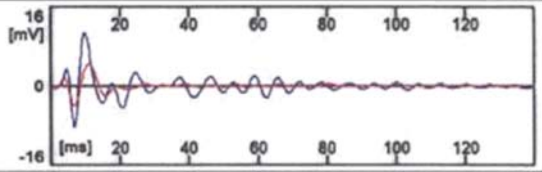
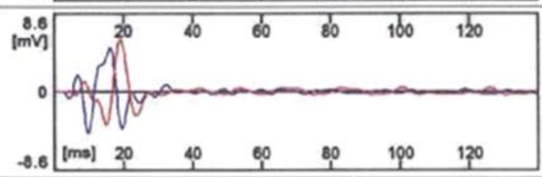
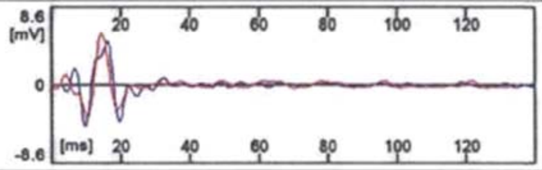
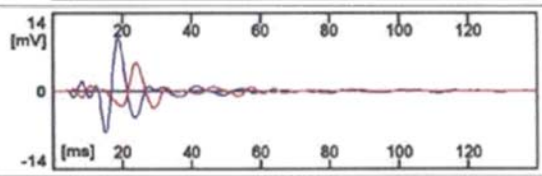
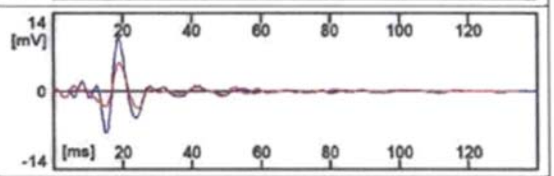
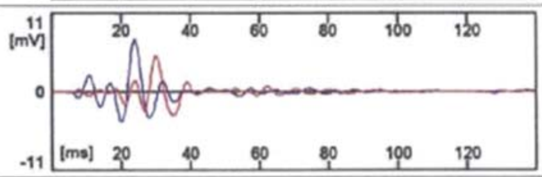
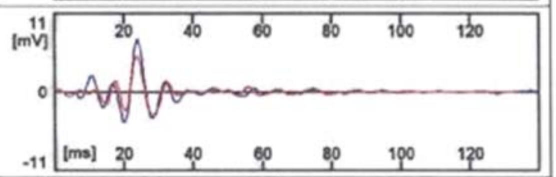
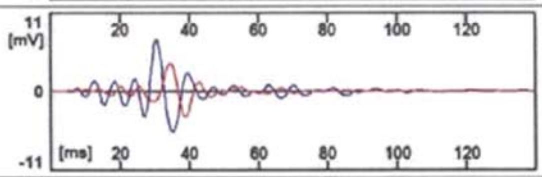
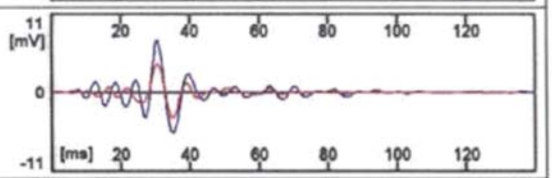
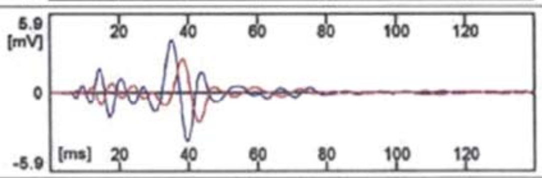
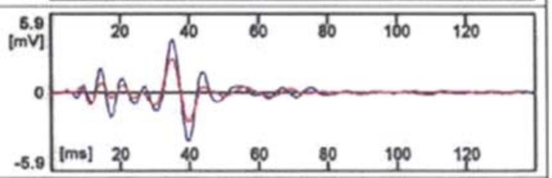
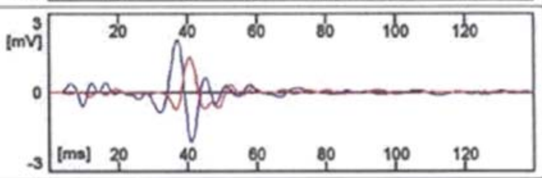
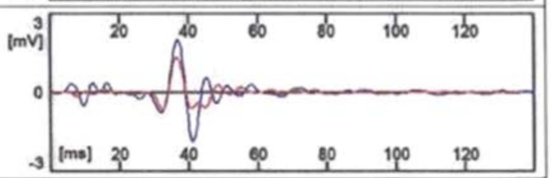
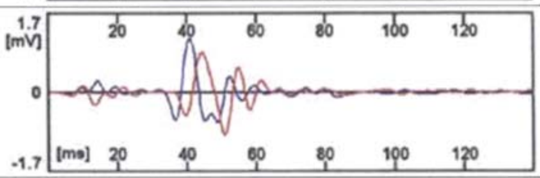
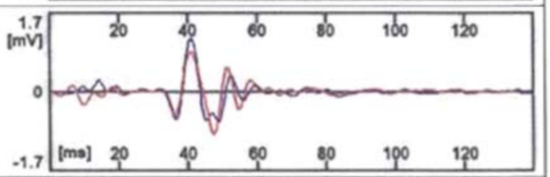
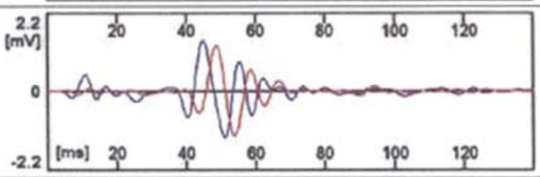
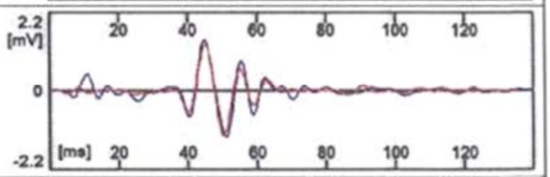
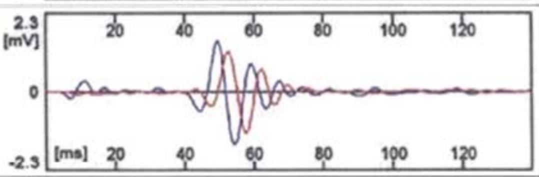
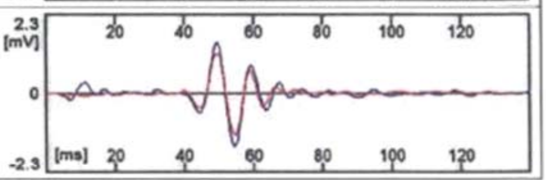
	RECORDED	RE-PHASED
Z = 5.50 m Ds = 0.50 m Dt = 2.97 ms Vs = 168 m/s 10 SEP 2015 10:53:33		
Z = 6.00 m Ds = 0.50 m Dt = 2.81 ms Vs = 177 m/s 10 SEP 2015 10:54:48		
Z = 6.50 m Ds = 0.50 m Dt = 2.58 ms Vs = 194 m/s 10 SEP 2015 10:56:32		
Z = 7.00 m Ds = 0.50 m Dt = 2.64 ms Vs = 189 m/s 10 SEP 2015 10:57:40		
Z = 7.50 m Ds = 0.50 m Dt = 2.15 ms Vs = 233 m/s 10 SEP 2015 10:59:23		
Z = 8.00 m Ds = 0.50 m Dt = 2.49 ms Vs = 200 m/s 10 SEP 2015 11:00:27		
Z = 8.50 m Ds = 0.50 m Dt = 2.20 ms Vs = 227 m/s 10 SEP 2015 11:02:05		
Z = 9.00 m Ds = 0.50 m Dt = 2.08 ms Vs = 240 m/s 10 SEP 2015 11:03:11		
Z = 9.50 m Ds = 0.50 m Dt = 2.33 ms Vs = 215 m/s 10 SEP 2015 11:04:57		
Z = 10.00 m Ds = 0.50 m Dt = 2.52 ms Vs = 199 m/s 10 SEP 2015 11:06:01		

VS-01 - Vs

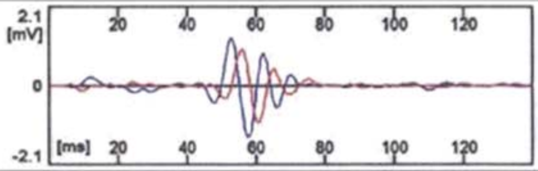
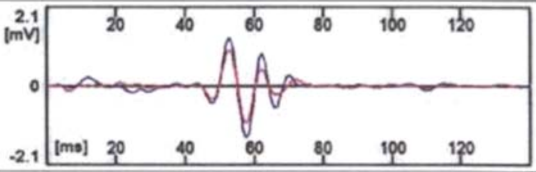
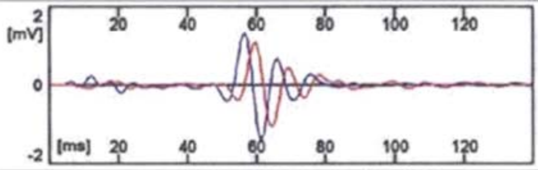
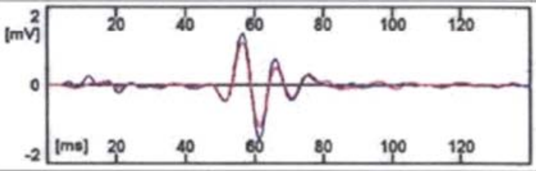
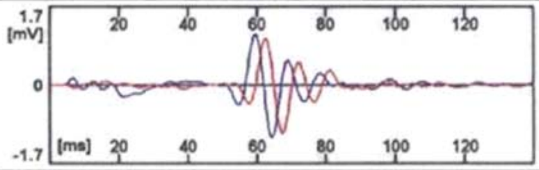
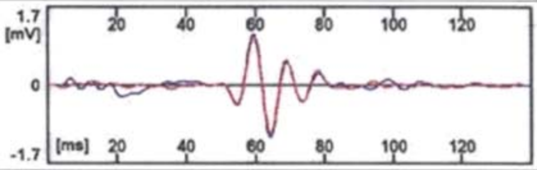
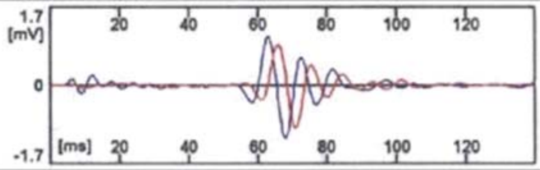
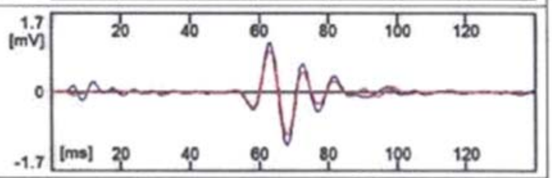
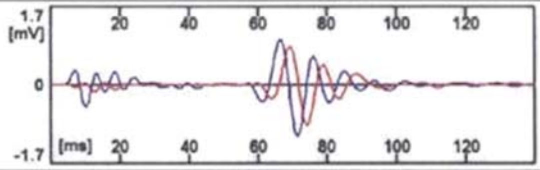
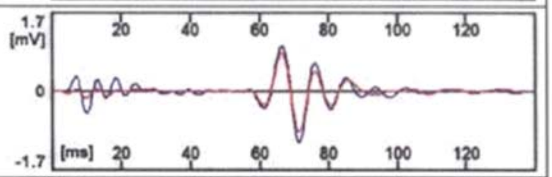
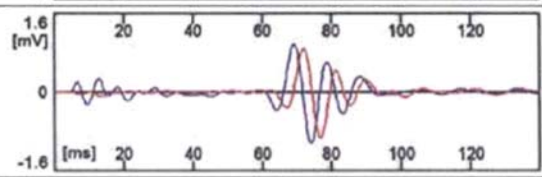
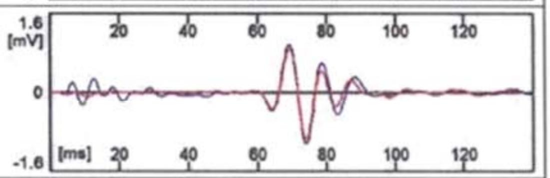
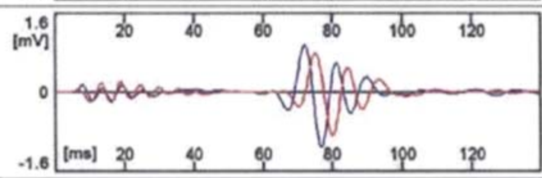
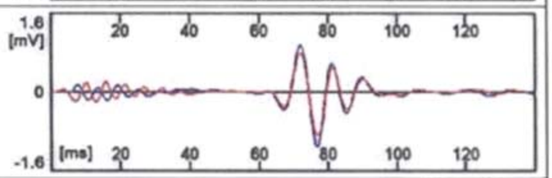
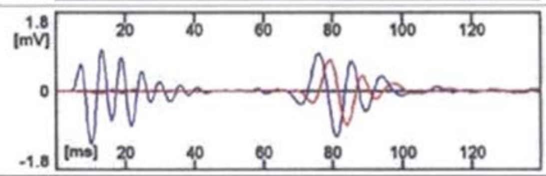
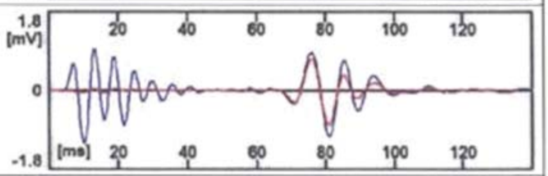
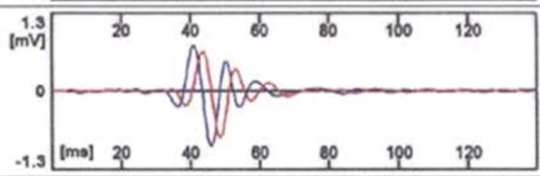
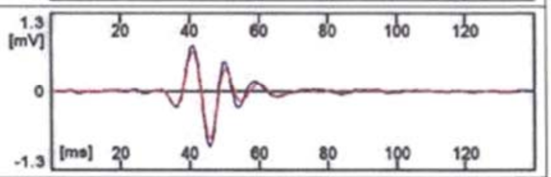
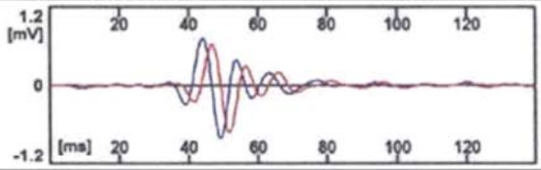
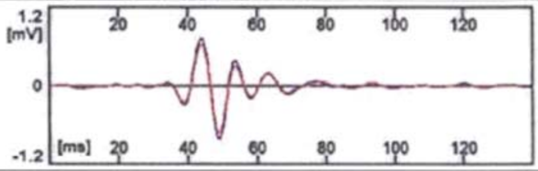
RECORDED **RE-PHASED**



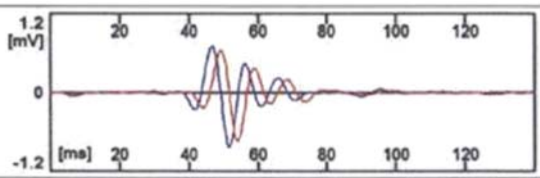
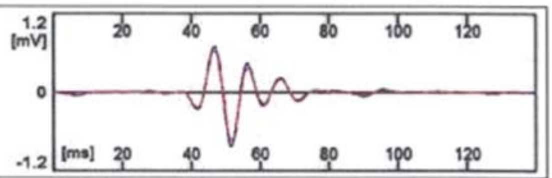
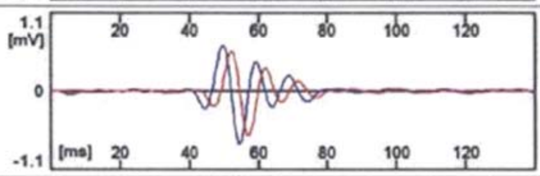
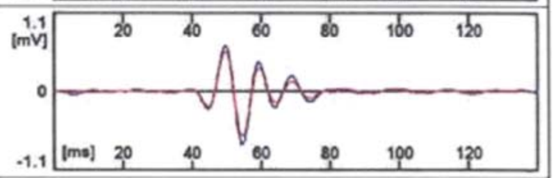
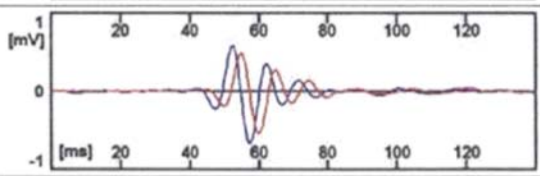
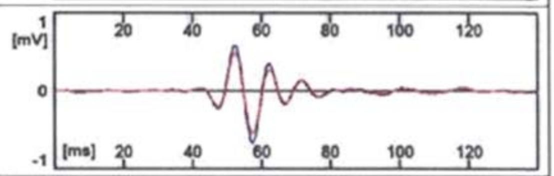
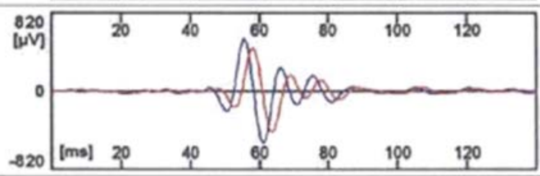
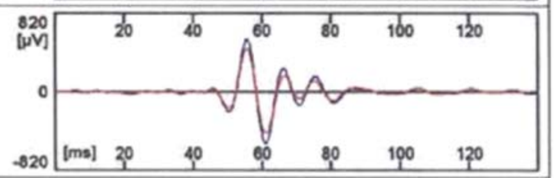
VS-02 - Vs

	RECORDED	RE-PHASED
Z = 0.50 m Ds = 0.42 m Dt = 3.12 ms Vs = 134 m/s 11 SEP 2015 11:58:50		
Z = 1.00 m Ds = 0.48 m Dt = 4.64 ms Vs = 103 m/s 11 SEP 2015 11:59:52		
Z = 1.50 m Ds = 0.49 m Dt = 5.00 ms Vs = 98 m/s 11 SEP 2015 12:02:11		
Z = 2.00 m Ds = 0.49 m Dt = 6.44 ms Vs = 77 m/s 11 SEP 2015 12:03:36		
Z = 2.50 m Ds = 0.50 m Dt = 3.83 ms Vs = 130 m/s 11 SEP 2015 12:05:52		
Z = 3.00 m Ds = 0.50 m Dt = 3.34 ms Vs = 149 m/s 11 SEP 2015 12:07:05		
Z = 3.50 m Ds = 0.50 m Dt = 4.03 ms Vs = 124 m/s 11 SEP 2015 12:08:54		
Z = 4.00 m Ds = 0.50 m Dt = 3.58 ms Vs = 139 m/s 11 SEP 2015 12:09:57		
Z = 4.50 m Ds = 0.50 m Dt = 3.60 ms Vs = 139 m/s 11 SEP 2015 12:11:41		
Z = 5.00 m Ds = 0.50 m Dt = 3.24 ms Vs = 154 m/s 11 SEP 2015 12:12:42		

VS-02 - Vs

	RECORDED	RE-PHASED
Z = 5.50 m Ds = 0.50 m Dt = 3.03 ms Vs = 165 m/s 11 SEP 2015 12:14:17		
Z = 6.00 m Ds = 0.50 m Dt = 3.03 ms Vs = 165 m/s 11 SEP 2015 12:15:21		
Z = 6.50 m Ds = 0.50 m Dt = 2.98 ms Vs = 167 m/s 11 SEP 2015 12:17:03		
Z = 7.00 m Ds = 0.50 m Dt = 2.89 ms Vs = 173 m/s 11 SEP 2015 12:18:15		
Z = 7.50 m Ds = 0.50 m Dt = 2.80 ms Vs = 178 m/s 11 SEP 2015 12:19:54		
Z = 8.00 m Ds = 0.50 m Dt = 2.80 ms Vs = 178 m/s 11 SEP 2015 12:20:45		
Z = 8.50 m Ds = 0.50 m Dt = 3.11 ms Vs = 161 m/s 11 SEP 2015 12:22:26		
Z = 9.00 m Ds = 0.50 m Dt = 3.18 ms Vs = 157 m/s 11 SEP 2015 12:23:24		
Z = 9.50 m Ds = 0.50 m Dt = 2.56 ms Vs = 195 m/s 11 SEP 2015 12:25:09		
Z = 10.00 m Ds = 0.50 m Dt = 2.63 ms Vs = 190 m/s 11 SEP 2015 12:25:58		

VS-02 - Vs

	RECORDED	RE-PHASED
Z = 10.50 m Ds = 0.50 m Dt = 2.48 ms Vs = 201 m/s 11 SEP 2015 12:27:23		
Z = 11.00 m Ds = 0.50 m Dt = 2.52 ms Vs = 199 m/s 11 SEP 2015 12:28:15		
Z = 11.50 m Ds = 0.50 m Dt = 2.48 ms Vs = 201 m/s 11 SEP 2015 12:29:40		
Z = 12.00 m Ds = 0.50 m Dt = 2.43 ms Vs = 205 m/s 11 SEP 2015 12:30:49		

VS-03 - Vs

RECORDED

RE-PHASED

Z = 0.50 m
Ds = 0.42 m
Dt = 4.08 ms
Vs = 102 m/s
10 SEP 2015
15:04:05

Z = 1.00 m
Ds = 0.48 m
Dt = 3.63 ms
Vs = 132 m/s
10 SEP 2015
15:05:07

Z = 1.50 m
Ds = 0.49 m
Dt = 4.37 ms
Vs = 112 m/s
10 SEP 2015
15:06:35

Z = 2.00 m
Ds = 0.49 m
Dt = 4.96 ms
Vs = 100 m/s
10 SEP 2015
15:07:55

Z = 2.50 m
Ds = 0.50 m
Dt = 4.64 ms
Vs = 107 m/s
10 SEP 2015
15:09:18

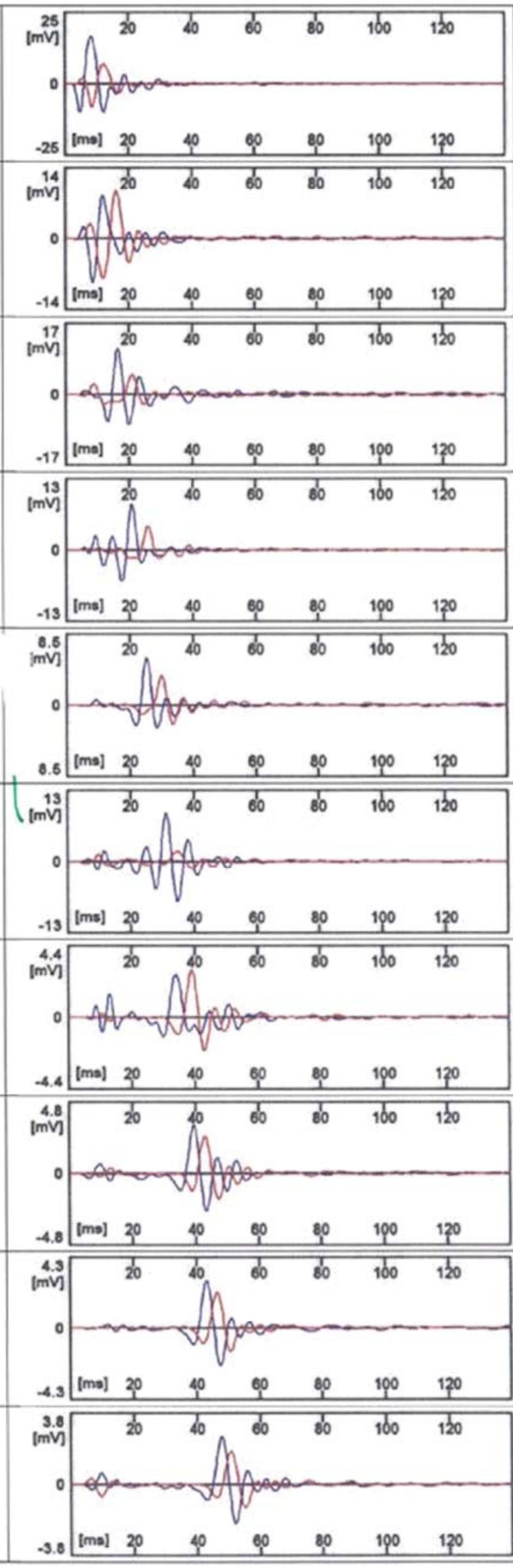
Z = 3.00 m
Ds = 0.50 m
Dt = 3.36 ms
Vs = 148 m/s
10 SEP 2015
15:10:24

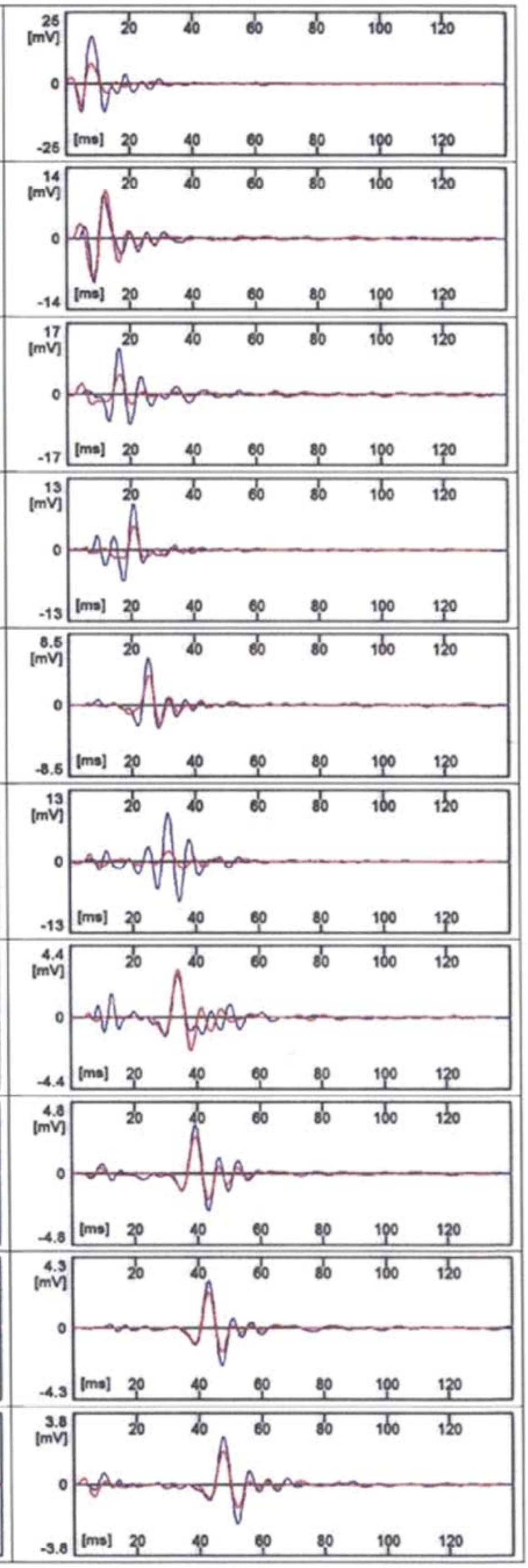
Z = 3.50 m
Ds = 0.50 m
Dt = 4.88 ms
Vs = 102 m/s
10 SEP 2015
15:12:05

Z = 4.00 m
Ds = 0.50 m
Dt = 3.78 ms
Vs = 132 m/s
10 SEP 2015
15:13:21

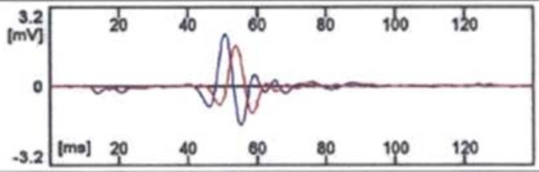
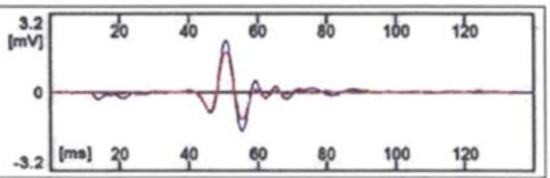
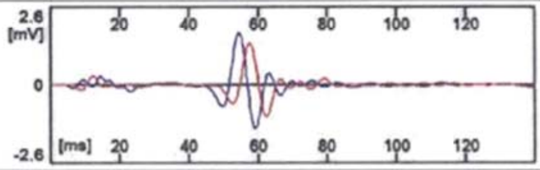
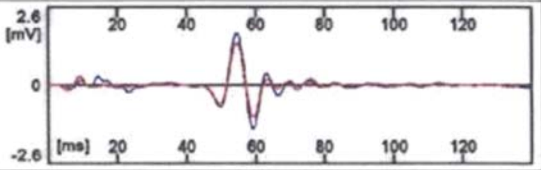
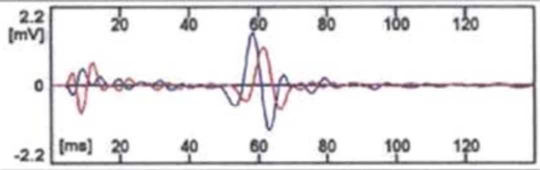
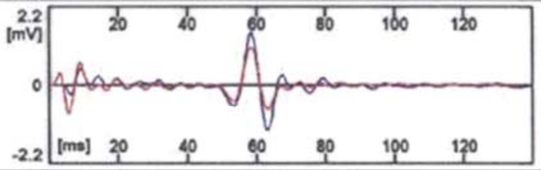
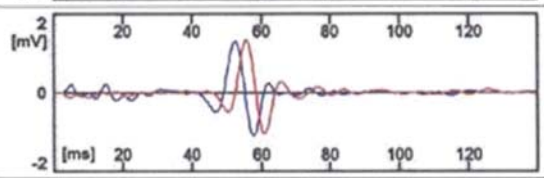
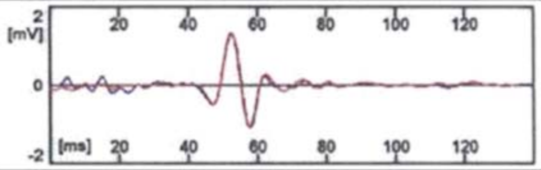
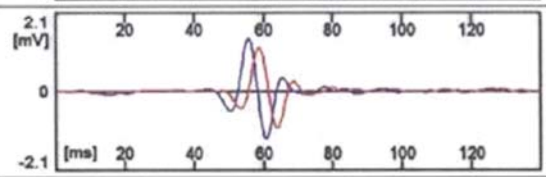
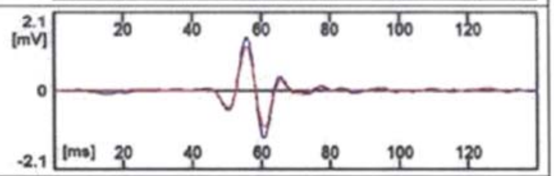
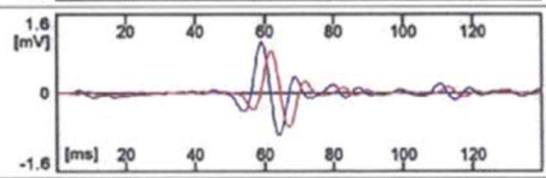
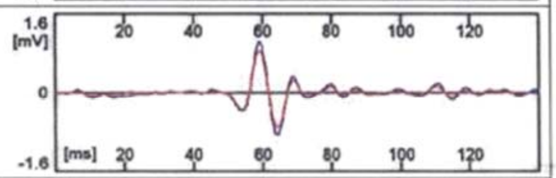
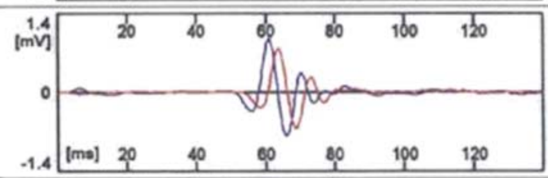
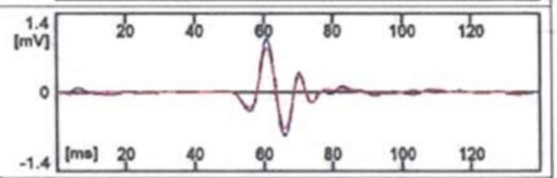
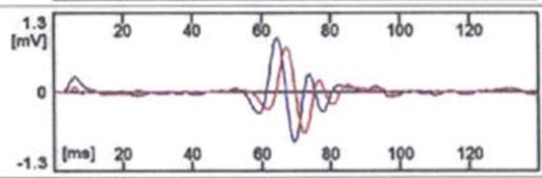
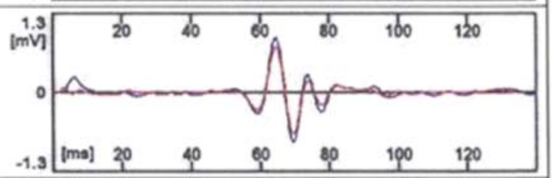
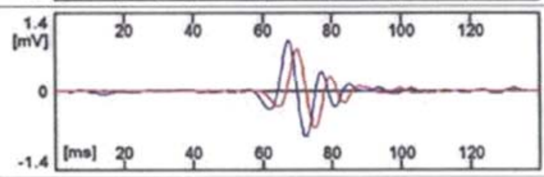
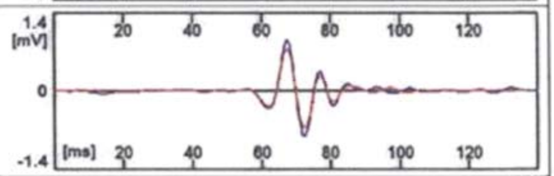
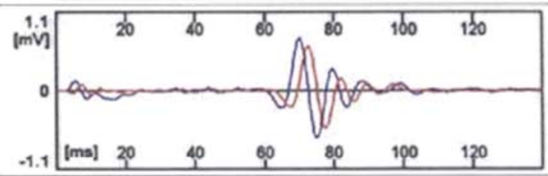
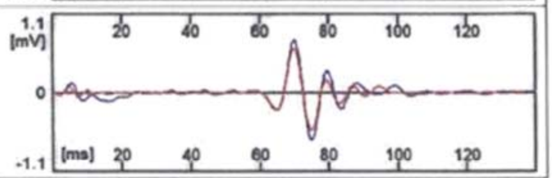
Z = 4.50 m
Ds = 0.50 m
Dt = 3.34 ms
Vs = 149 m/s
10 SEP 2015
15:14:34

Z = 5.00 m
Ds = 0.50 m
Dt = 3.05 ms
Vs = 164 m/s
10 SEP 2015
15:15:42





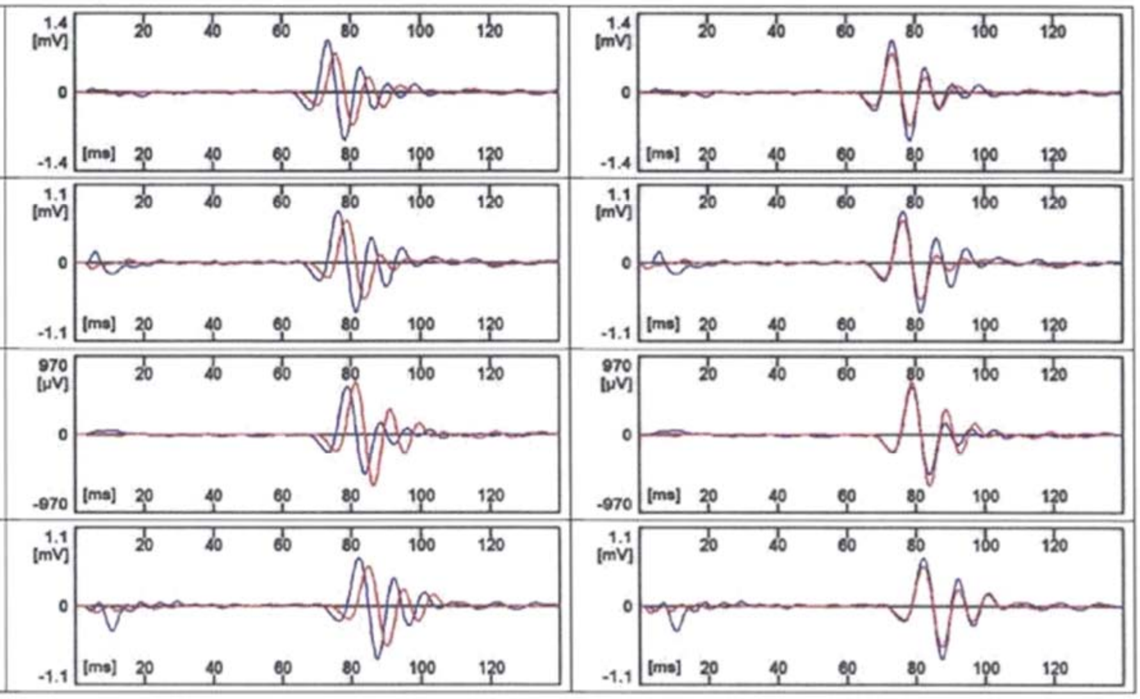
VS-03 - Vs

	RECORDED	RE-PHASED
Z = 5.50 m Ds = 0.50 m Dt = 3.20 ms Vs = 156 m/s 10 SEP 2015 15:16:43		
Z = 6.00 m Ds = 0.50 m Dt = 3.11 ms Vs = 161 m/s 10 SEP 2015 15:17:58		
Z = 6.50 m Ds = 0.50 m Dt = 3.07 ms Vs = 163 m/s 10 SEP 2015 15:19:09		
Z = 7.00 m Ds = 0.50 m Dt = 3.34 ms Vs = 150 m/s 10 SEP 2015 15:20:16		
Z = 7.50 m Ds = 0.50 m Dt = 3.02 ms Vs = 166 m/s 10 SEP 2015 15:21:17		
Z = 8.00 m Ds = 0.50 m Dt = 2.79 ms Vs = 179 m/s 10 SEP 2015 15:22:24		
Z = 8.50 m Ds = 0.50 m Dt = 2.70 ms Vs = 185 m/s 10 SEP 2015 15:23:23		
Z = 9.00 m Ds = 0.50 m Dt = 2.73 ms Vs = 183 m/s 10 SEP 2015 15:24:35		
Z = 9.50 m Ds = 0.50 m Dt = 2.52 ms Vs = 198 m/s 10 SEP 2015 15:25:40		
Z = 10.00 m Ds = 0.50 m Dt = 2.55 ms Vs = 196 m/s 10 SEP 2015 15:26:41		

VS-03 - Vs

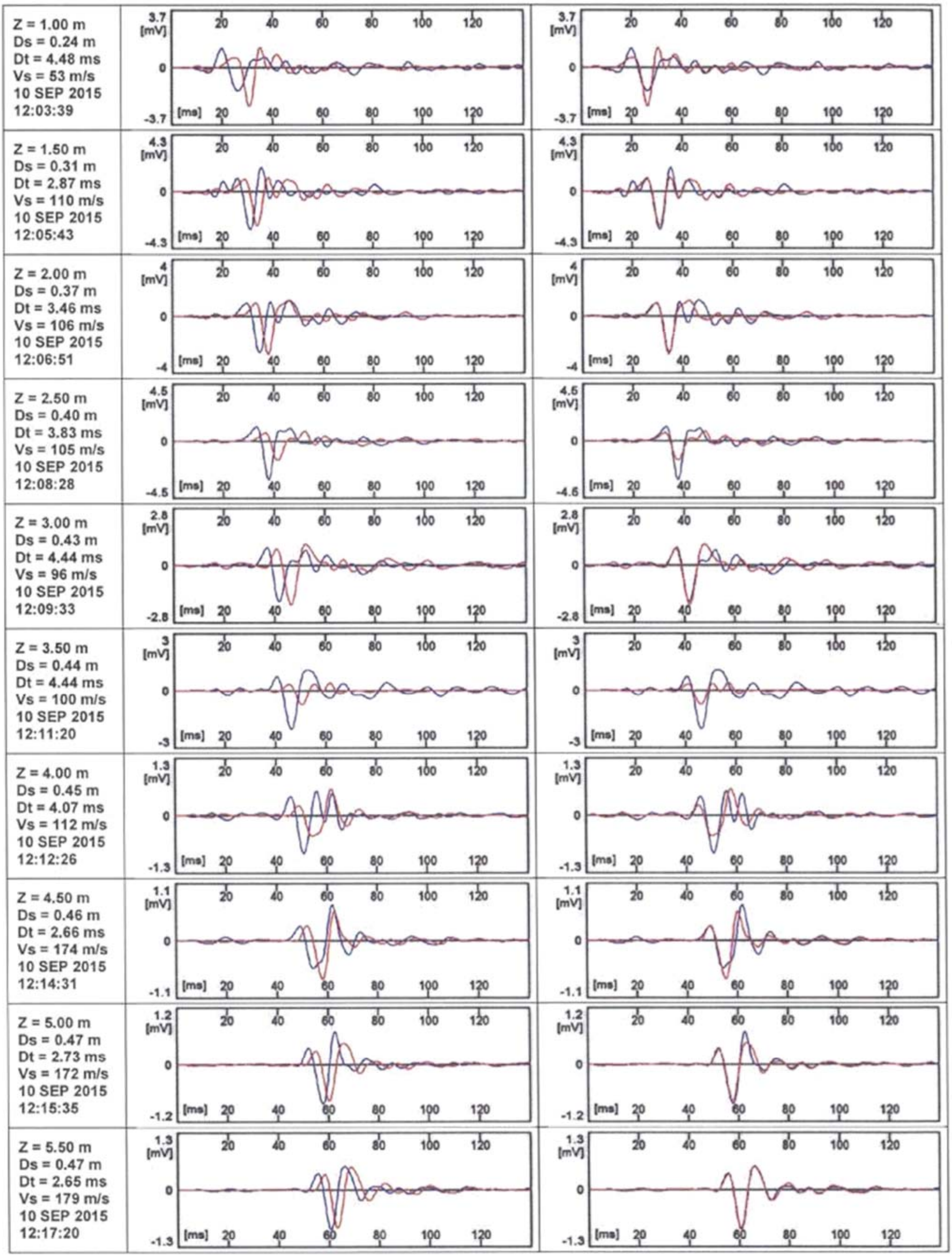
RECORDED | RE-PHASED

Z = 10.50 m Ds = 0.50 m Dt = 2.24 ms Vs = 223 m/s 10 SEP 2015 15:27:42	
Z = 11.00 m Ds = 0.50 m Dt = 2.46 ms Vs = 203 m/s 10 SEP 2015 15:28:50	
Z = 11.50 m Ds = 0.50 m Dt = 2.46 ms Vs = 203 m/s 10 SEP 2015 15:29:48	
Z = 12.00 m Ds = 0.50 m Dt = 2.77 ms Vs = 180 m/s 10 SEP 2015 15:30:48	



VS-04 - Vs

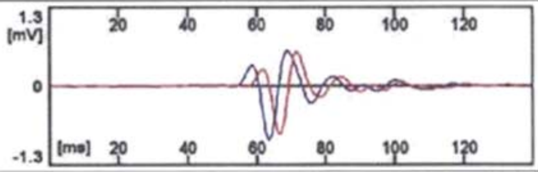
RECORDED **RE-PHASED**

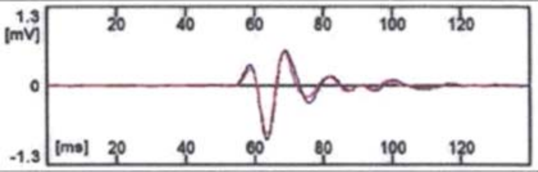


VS-04 - Vs

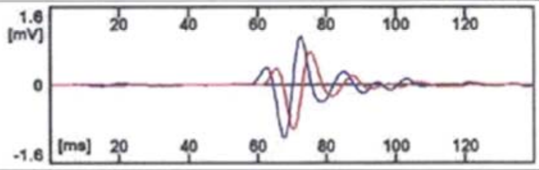
RECORDED | **RE-PHASED**

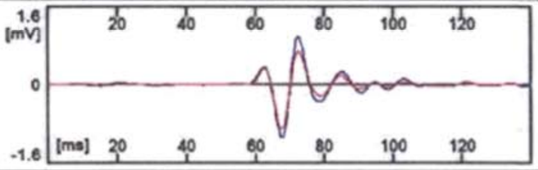
Z = 6.00 m
Ds = 0.48 m
Dt = 2.98 ms
Vs = 160 m/s
10 SEP 2015
12:18:27



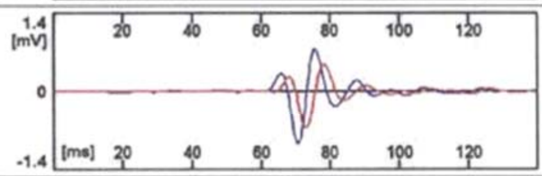


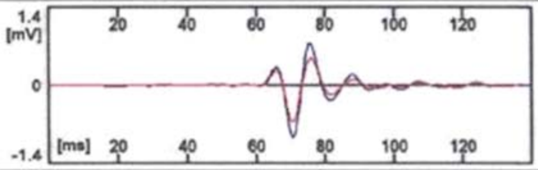
Z = 6.50 m
Ds = 0.48 m
Dt = 2.73 ms
Vs = 176 m/s
10 SEP 2015
12:20:10



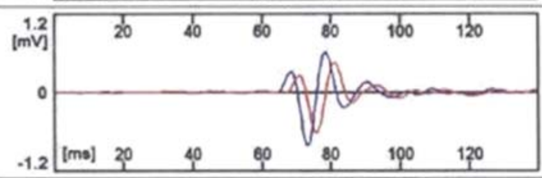


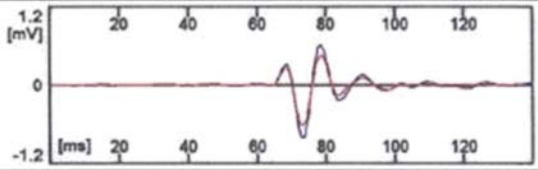
Z = 7.00 m
Ds = 0.48 m
Dt = 2.32 ms
Vs = 209 m/s
10 SEP 2015
12:21:16



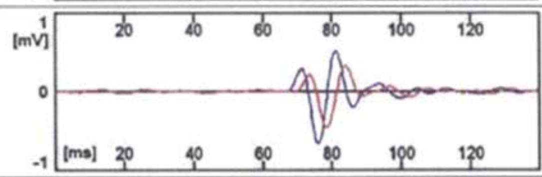


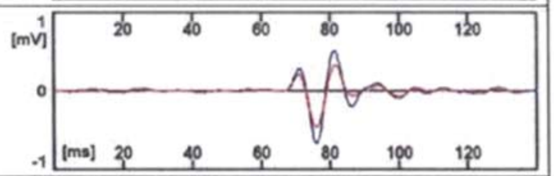
Z = 7.50 m
Ds = 0.49 m
Dt = 2.51 ms
Vs = 193 m/s
10 SEP 2015
12:23:00



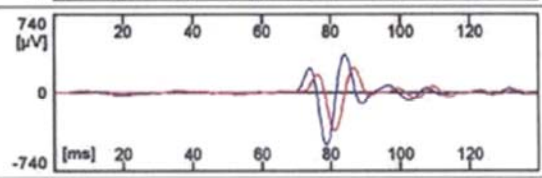


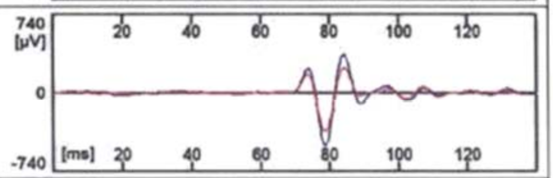
Z = 8.00 m
Ds = 0.49 m
Dt = 2.47 ms
Vs = 197 m/s
10 SEP 2015
12:24:11



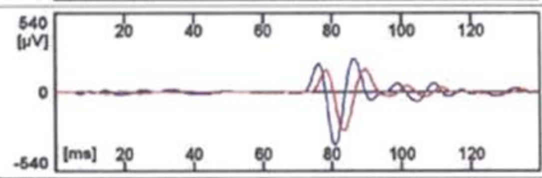


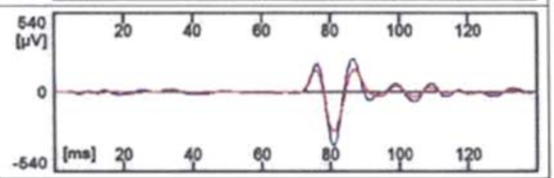
Z = 8.50 m
Ds = 0.49 m
Dt = 2.39 ms
Vs = 205 m/s
10 SEP 2015
12:25:58



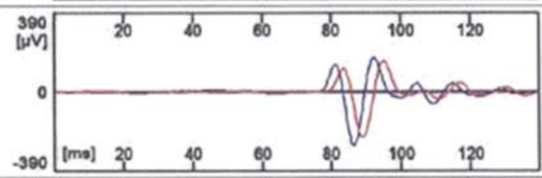


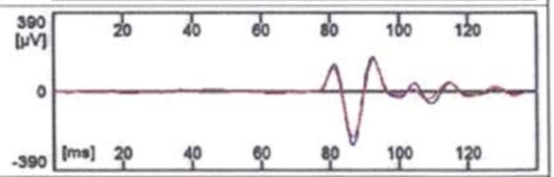
Z = 9.00 m
Ds = 0.49 m
Dt = 2.46 ms
Vs = 199 m/s
10 SEP 2015
12:27:11



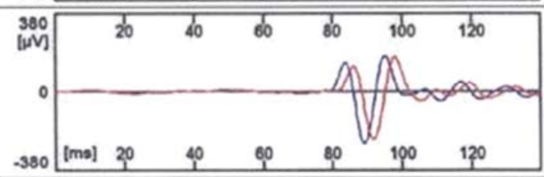


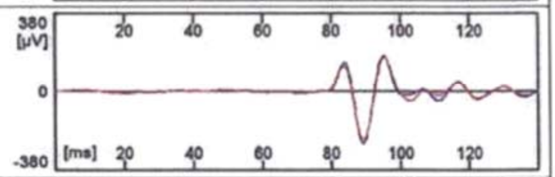
Z = 10.00 m
Ds = 0.49 m
Dt = 2.51 ms
Vs = 196 m/s
10 SEP 2015
12:30:37



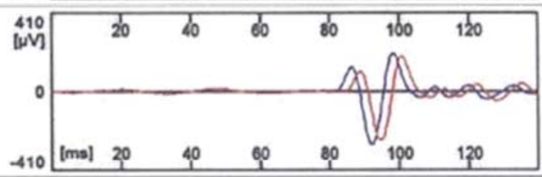


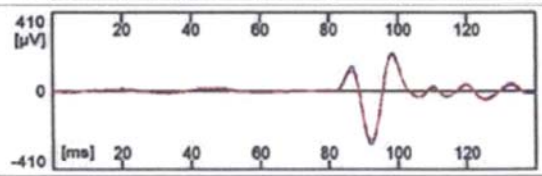
Z = 10.50 m
Ds = 0.49 m
Dt = 2.56 ms
Vs = 192 m/s
10 SEP 2015
12:32:45



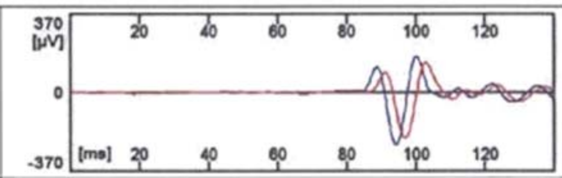
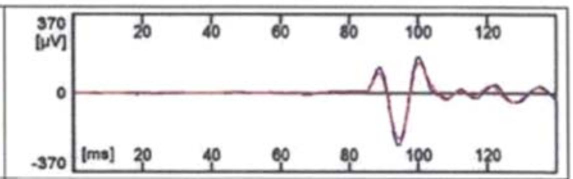
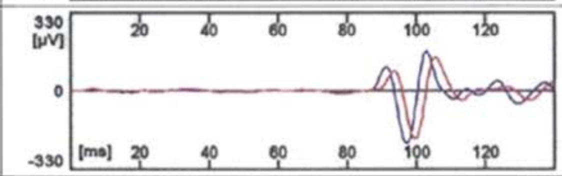
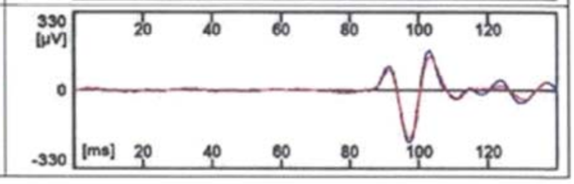


Z = 11.00 m
Ds = 0.49 m
Dt = 2.50 ms
Vs = 197 m/s
10 SEP 2015
12:33:57





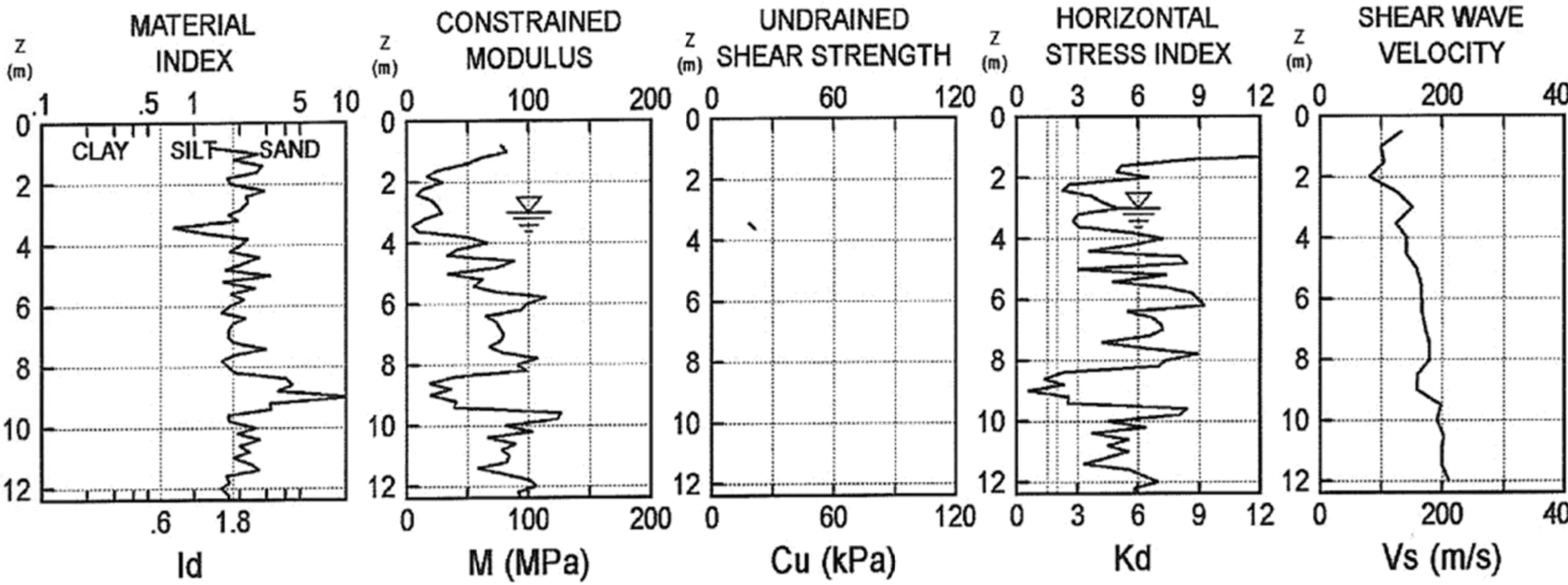
VS-04 - Vs

	RECORDED	RE-PHASED
Z = 11.50 m Ds = 0.49 m Dt = 2.56 ms Vs = 193 m/s 10 SEP 2015 12:35:51		
Z = 12.00 m Ds = 0.49 m Dt = 2.36 ms Vs = 210 m/s 10 SEP 2015 12:36:57		

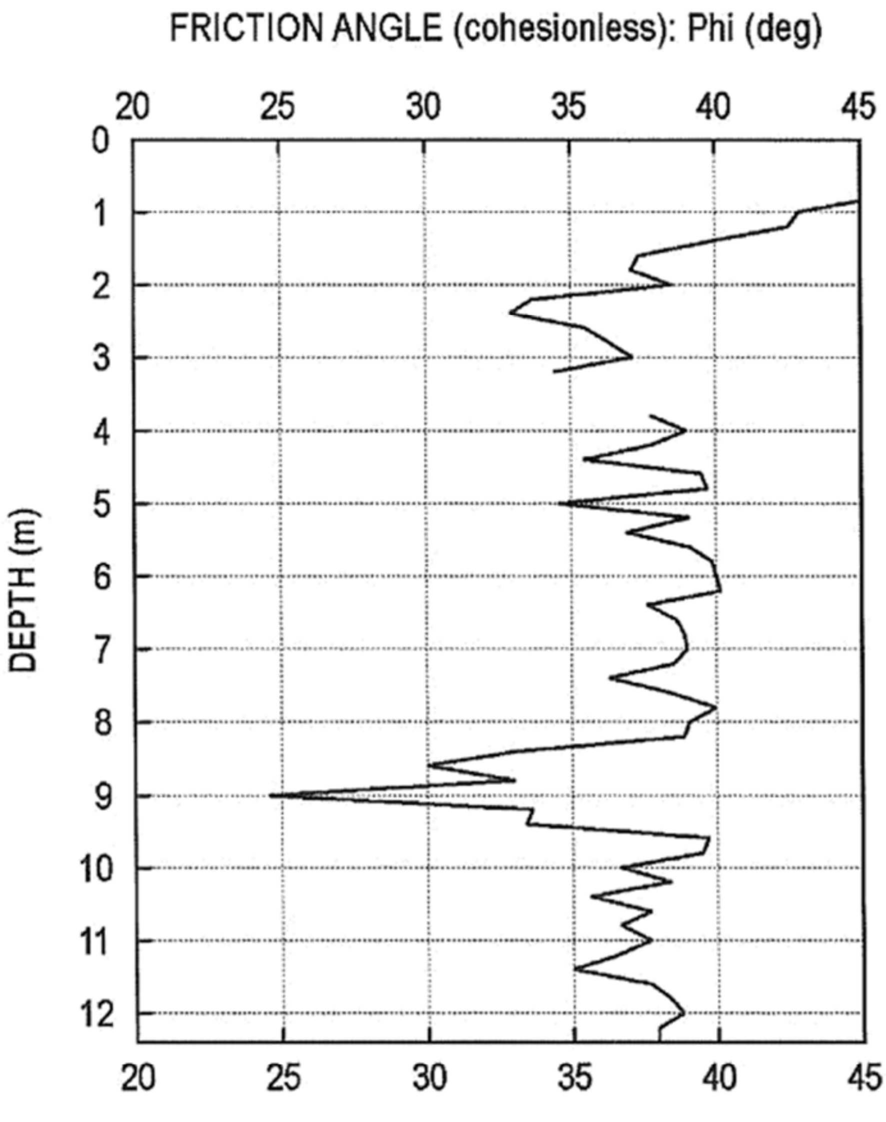
Appendix L – Research Post DSM DMT and sDMT Results

SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-X** 11 SEP 2015



Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-X**
INTERPRETED GEOTECHNICAL PARAMETERS		11 SEP 2015



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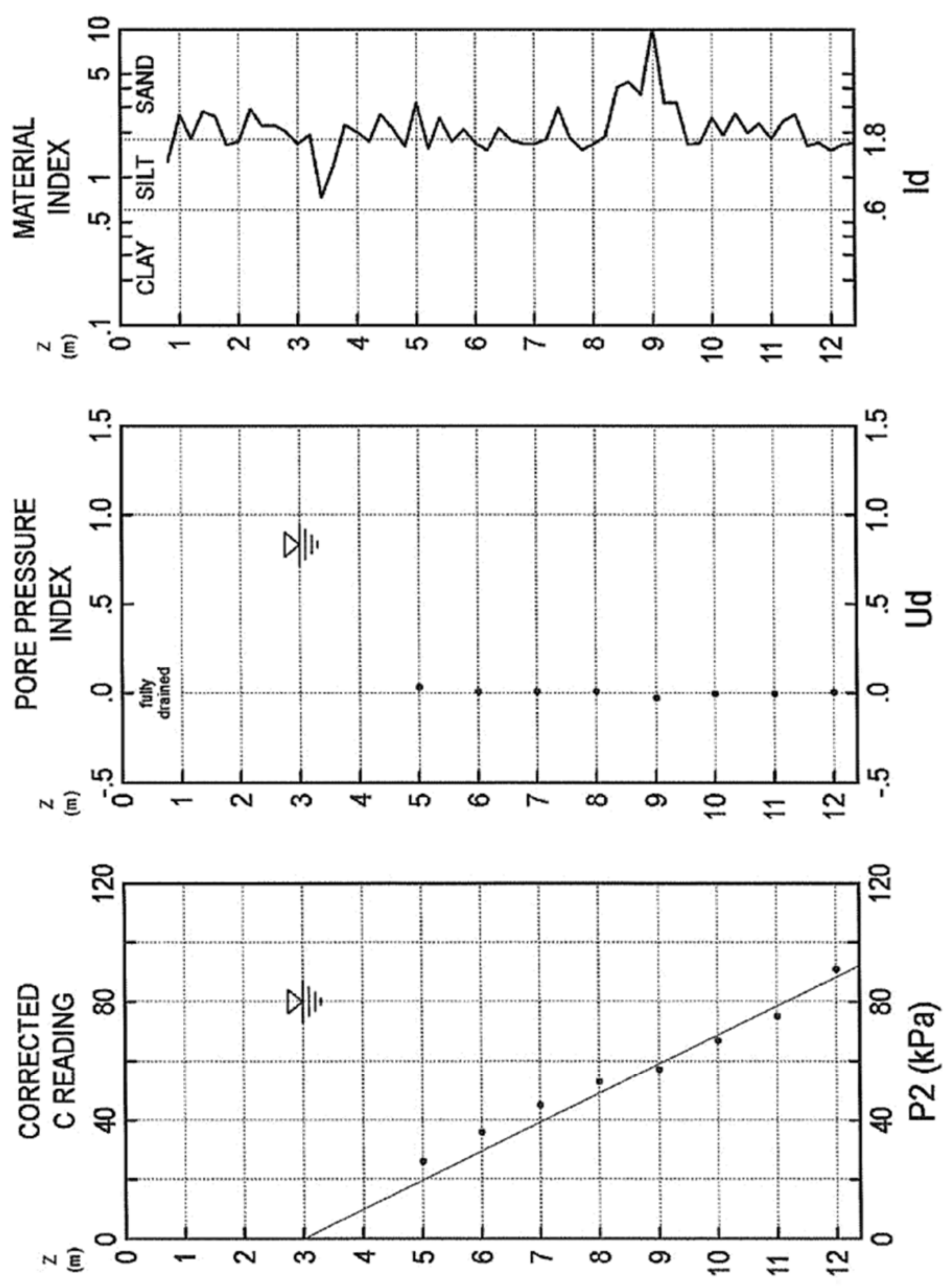
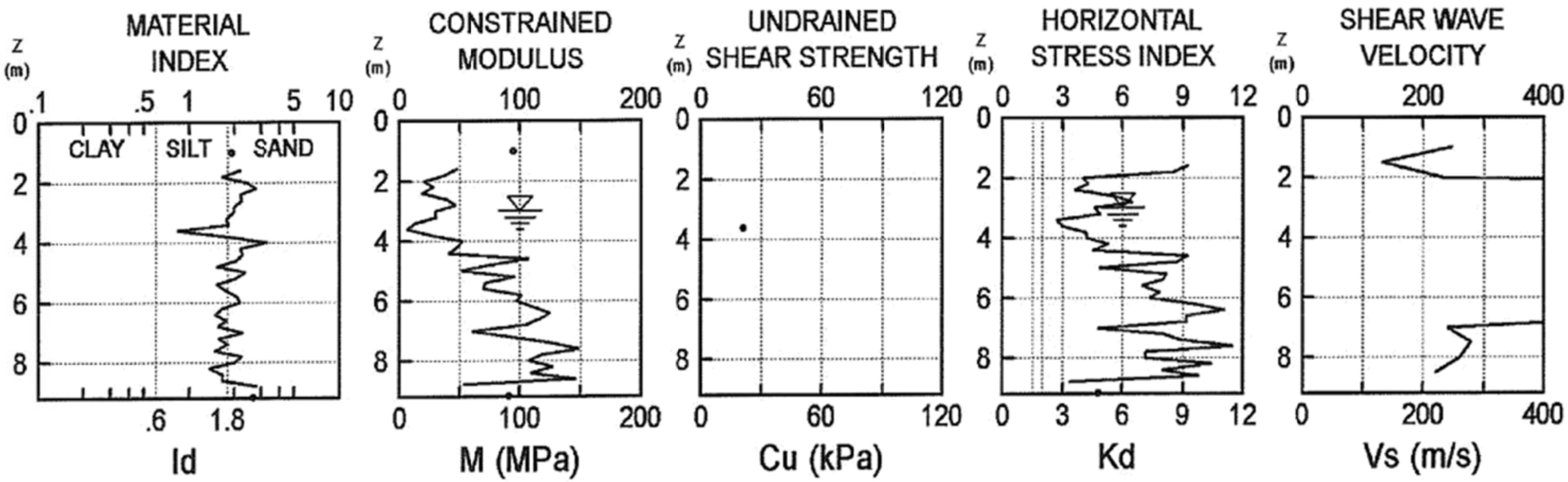
DILATOMETER TEST (DMT)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X
INTERPRETED GEOTECHNICAL PARAMETERS		11 SEP 2015

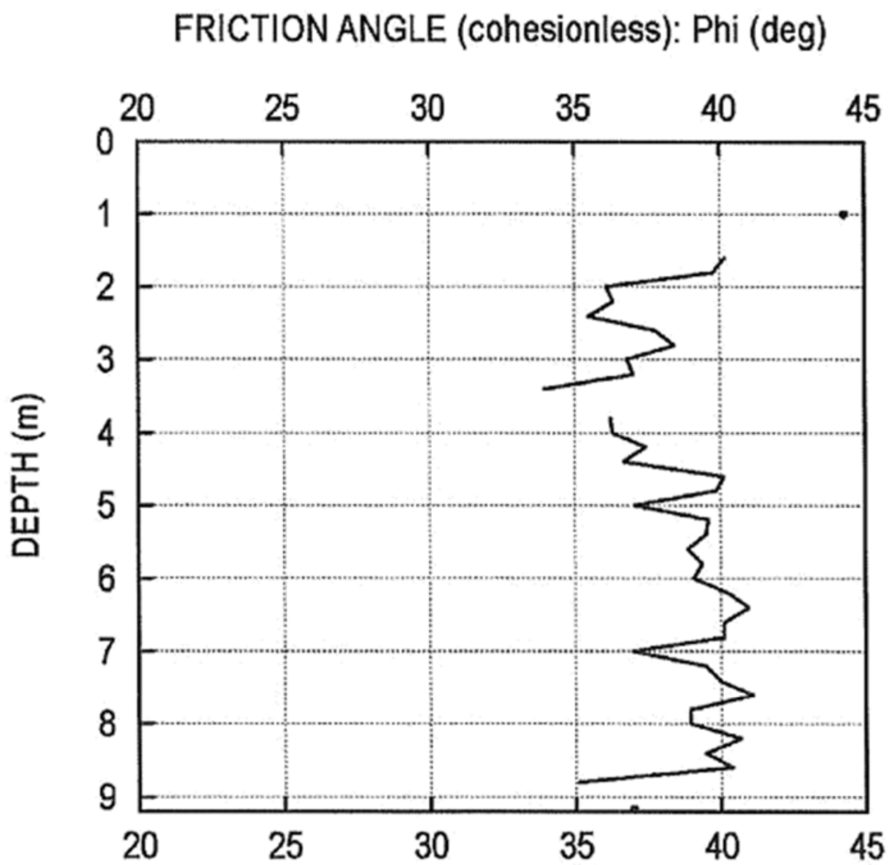


SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST SDMT-X500 12 JAN 2016

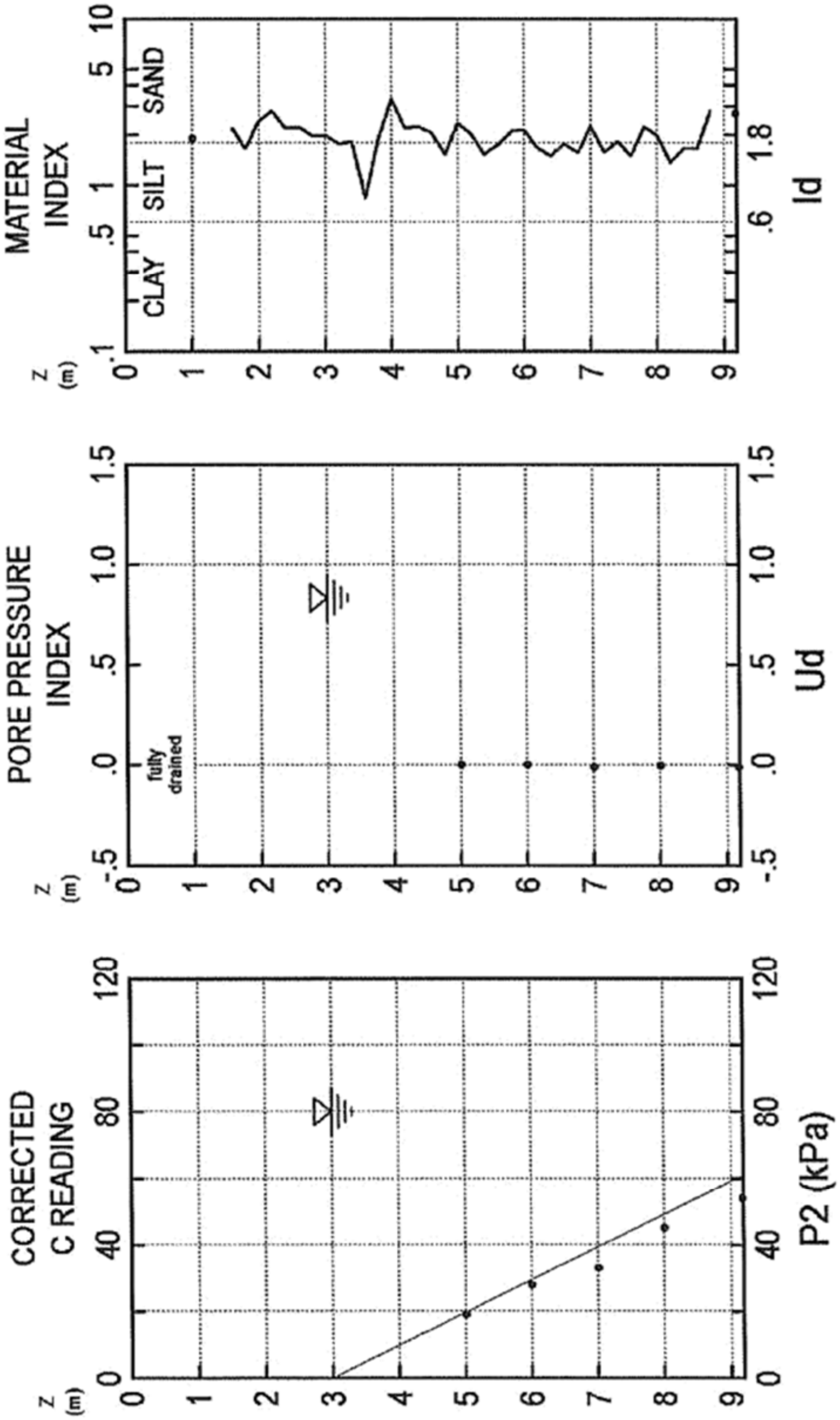


Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-X500**
INTERPRETED GEOTECHNICAL PARAMETERS		12 JAN 2016



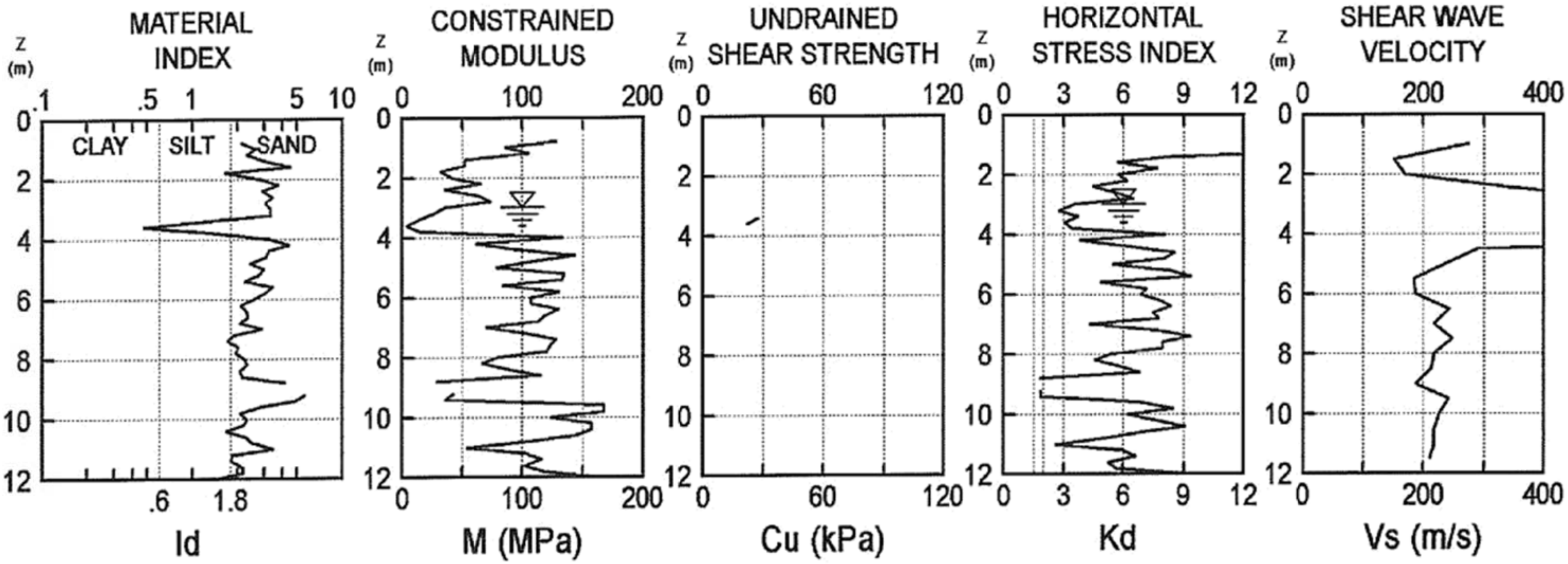
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X500
INTERPRETED GEOTECHNICAL PARAMETERS		12 JAN 2016



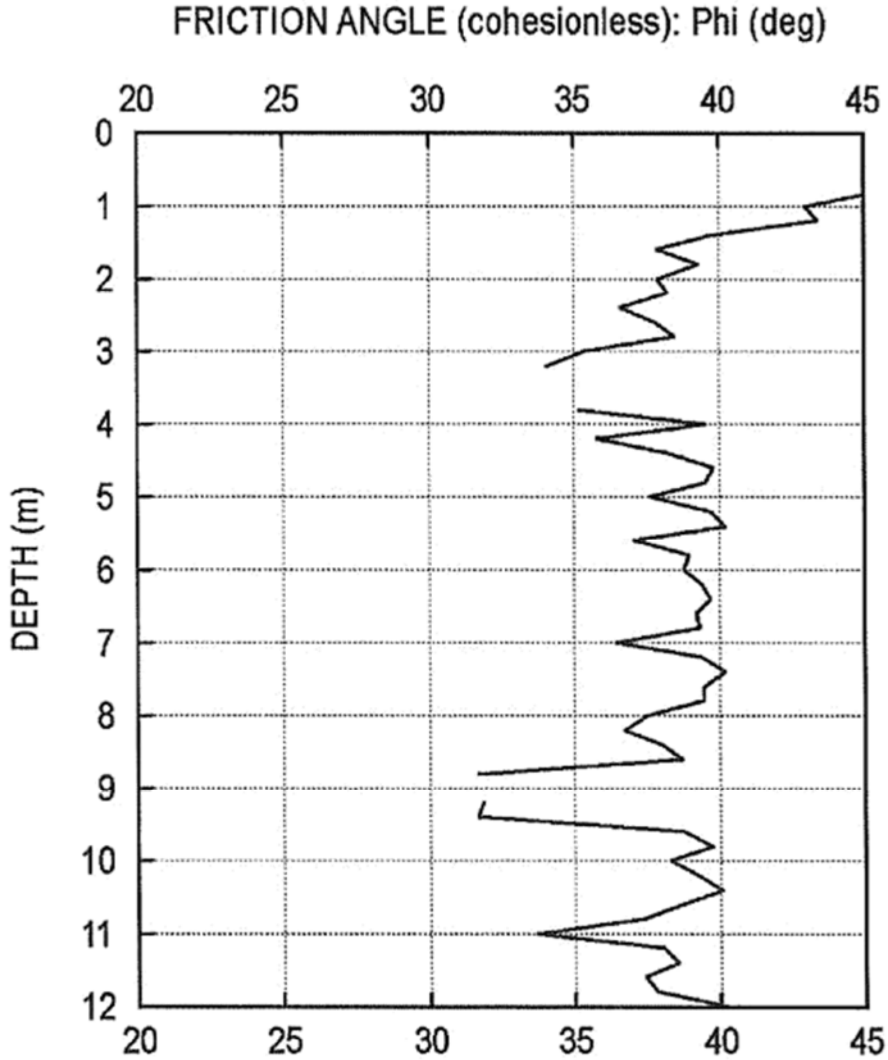
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST SDMT-X750 14 JAN 2016



DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X750
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016



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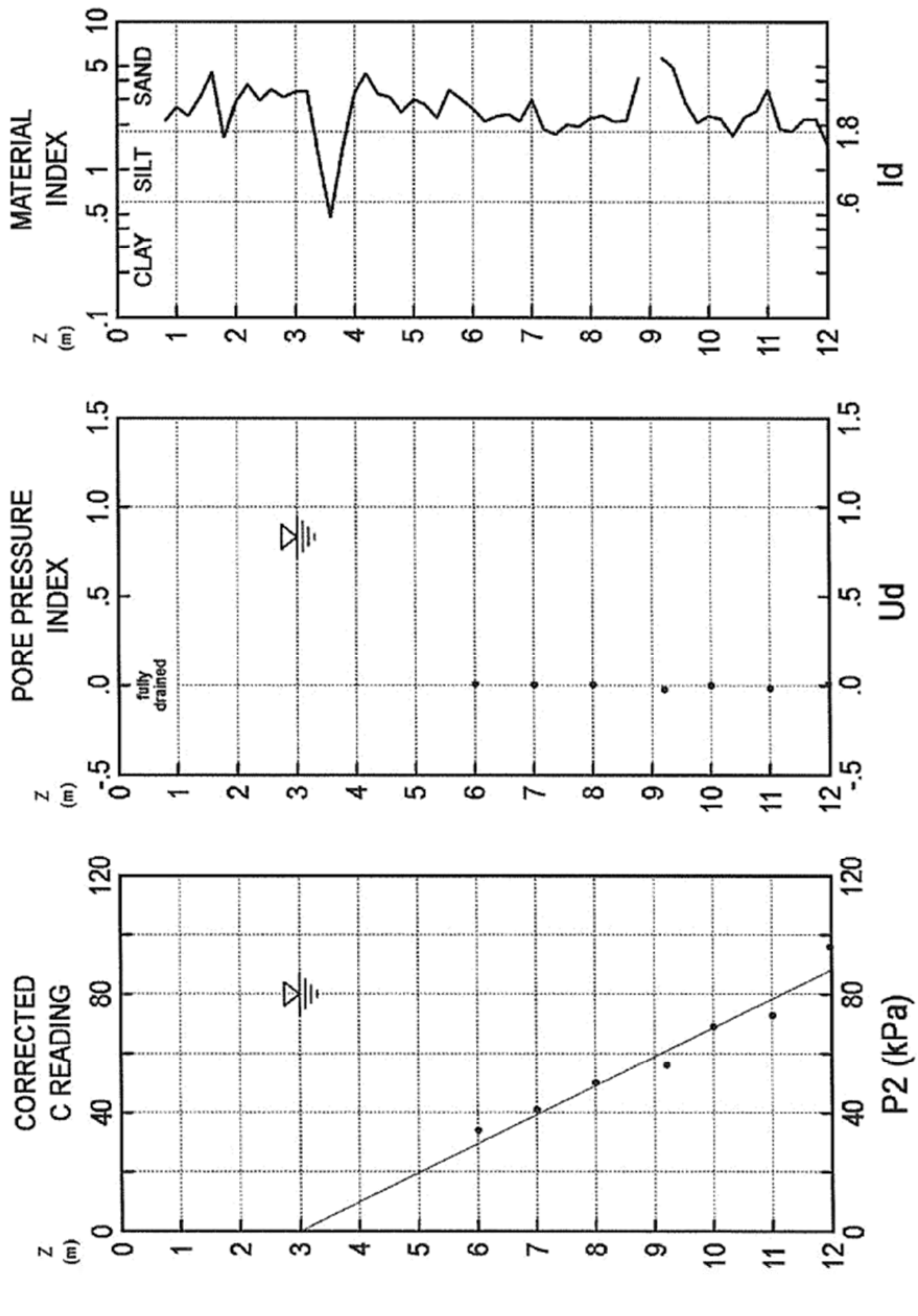
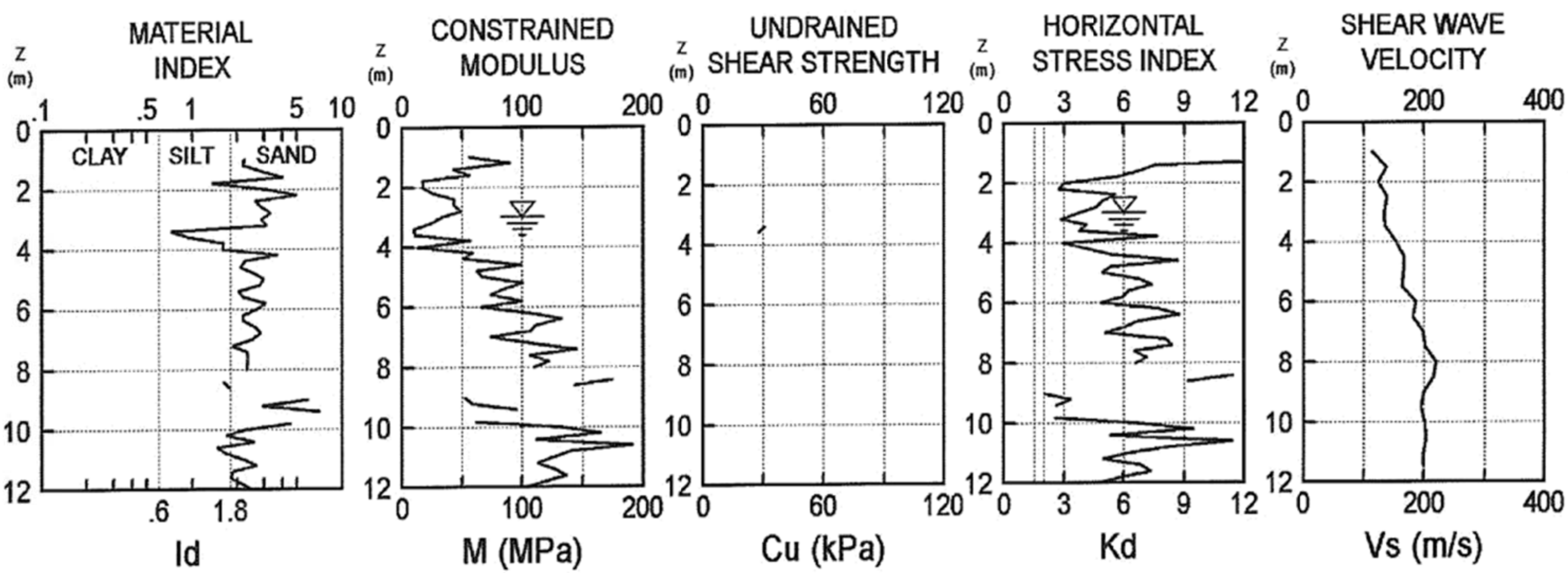
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X750
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016



SEISMIC DILATOMETER TEST (S D M T)

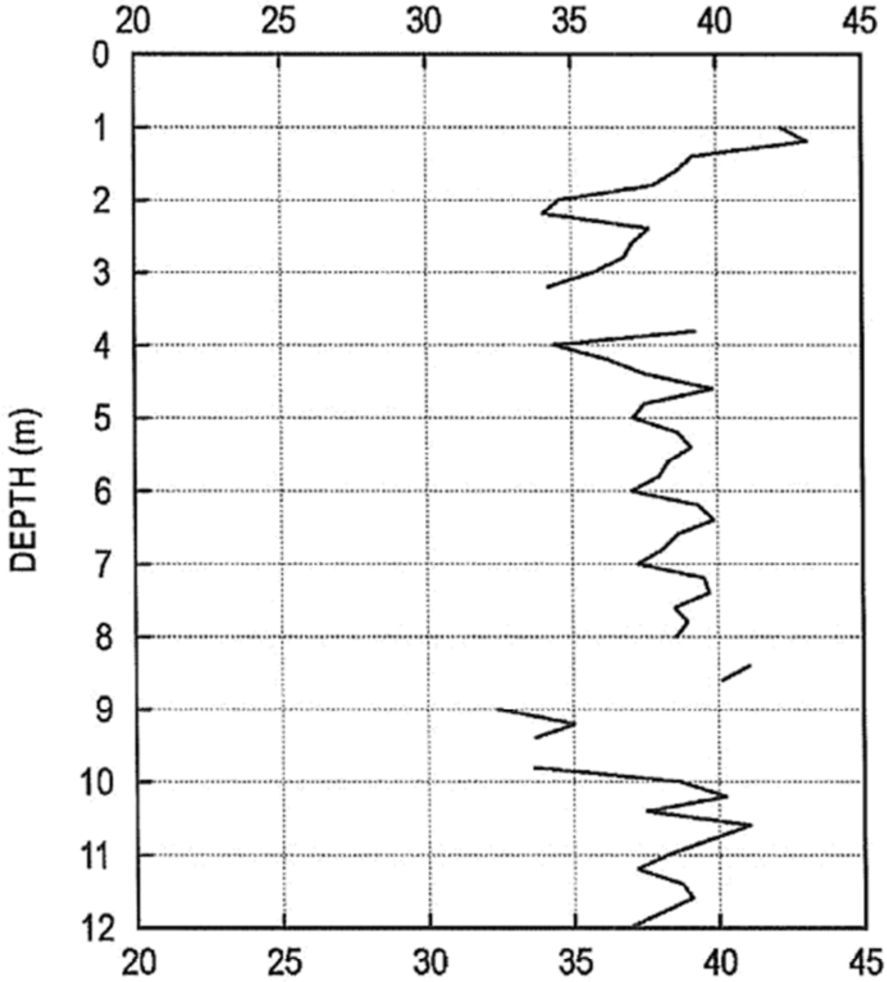
Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST SDMT-X1000 15 JAN 2016



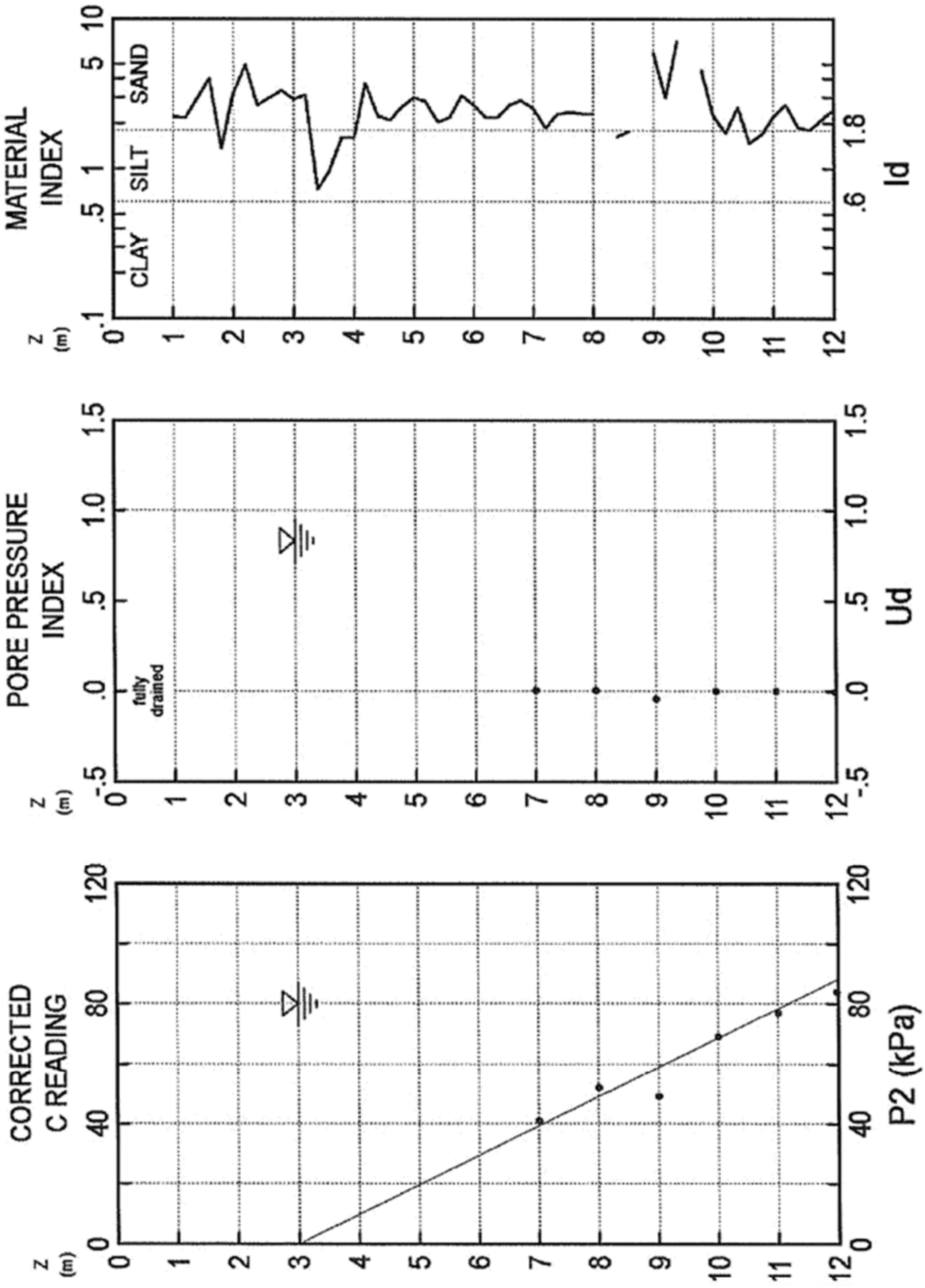
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X1000
INTERPRETED GEOTECHNICAL PARAMETERS		15 JAN 2016

FRICTION ANGLE (cohesionless): Phi (deg)

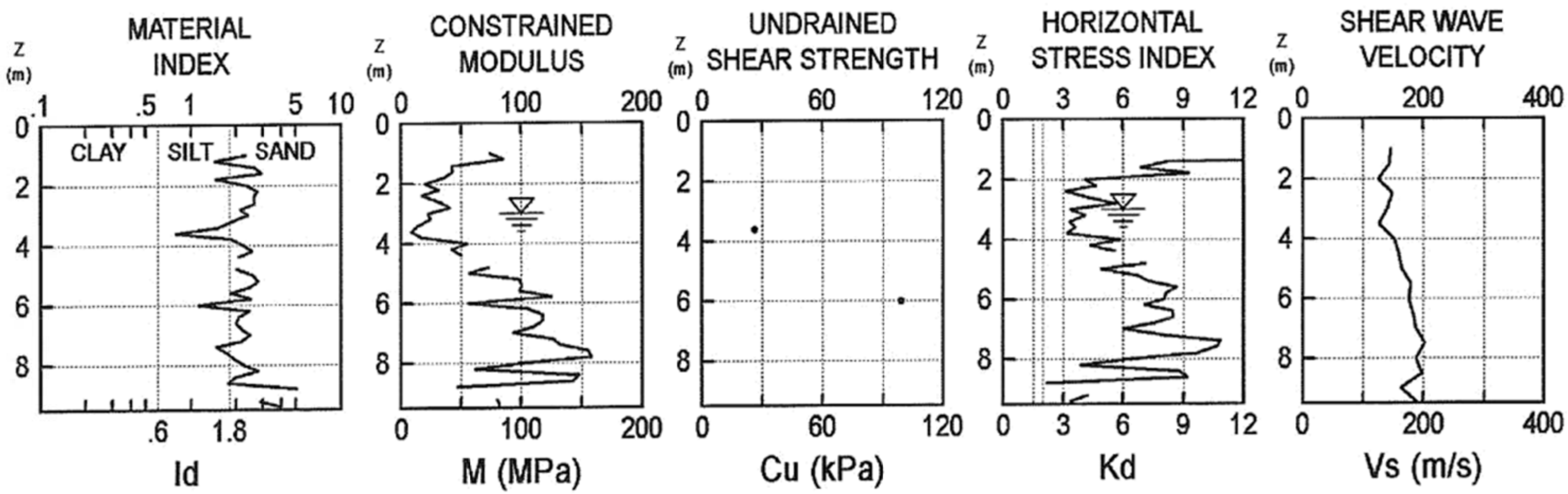


Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X1000
INTERPRETED GEOTECHNICAL PARAMETERS		15 JAN 2016

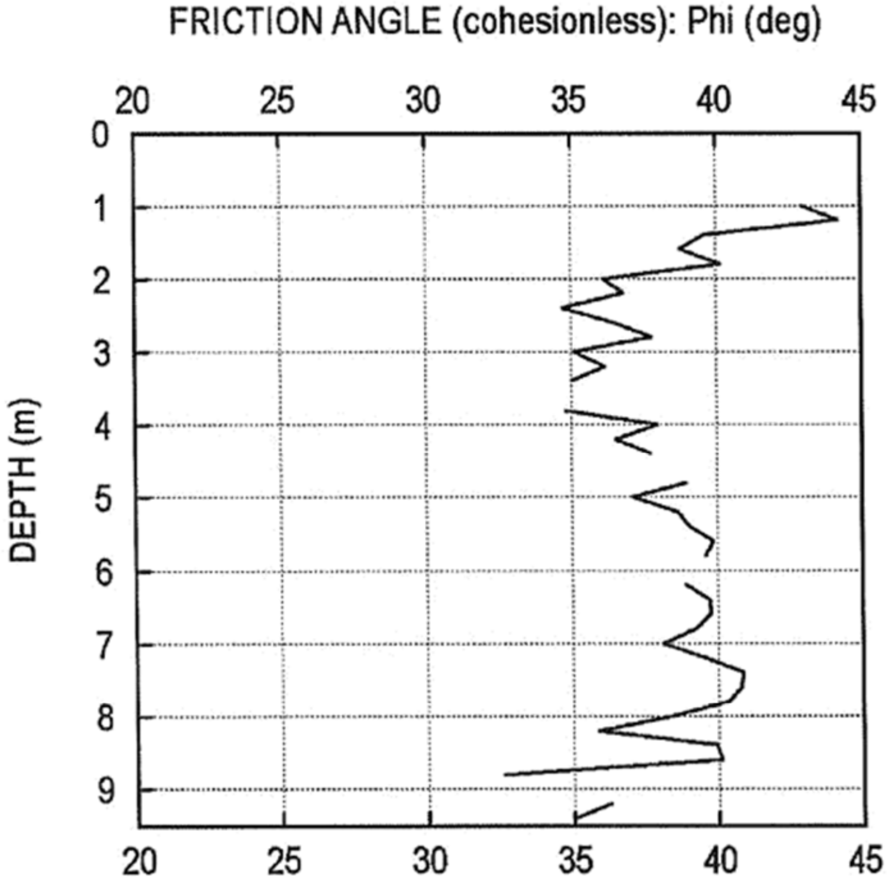


SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-X1250** 12 JAN 2016



Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X1250
INTERPRETED GEOTECHNICAL PARAMETERS		12 JAN 2016



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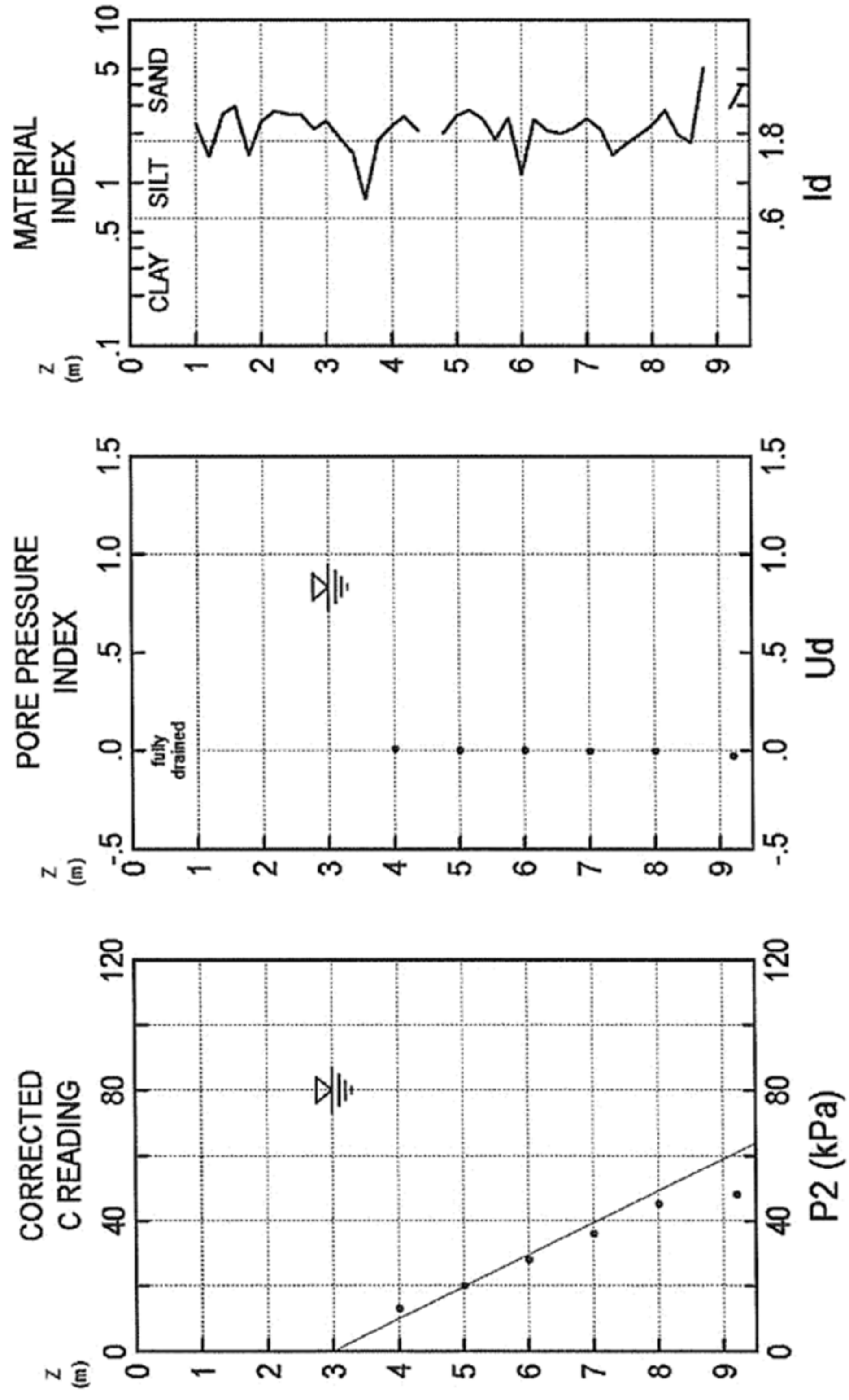
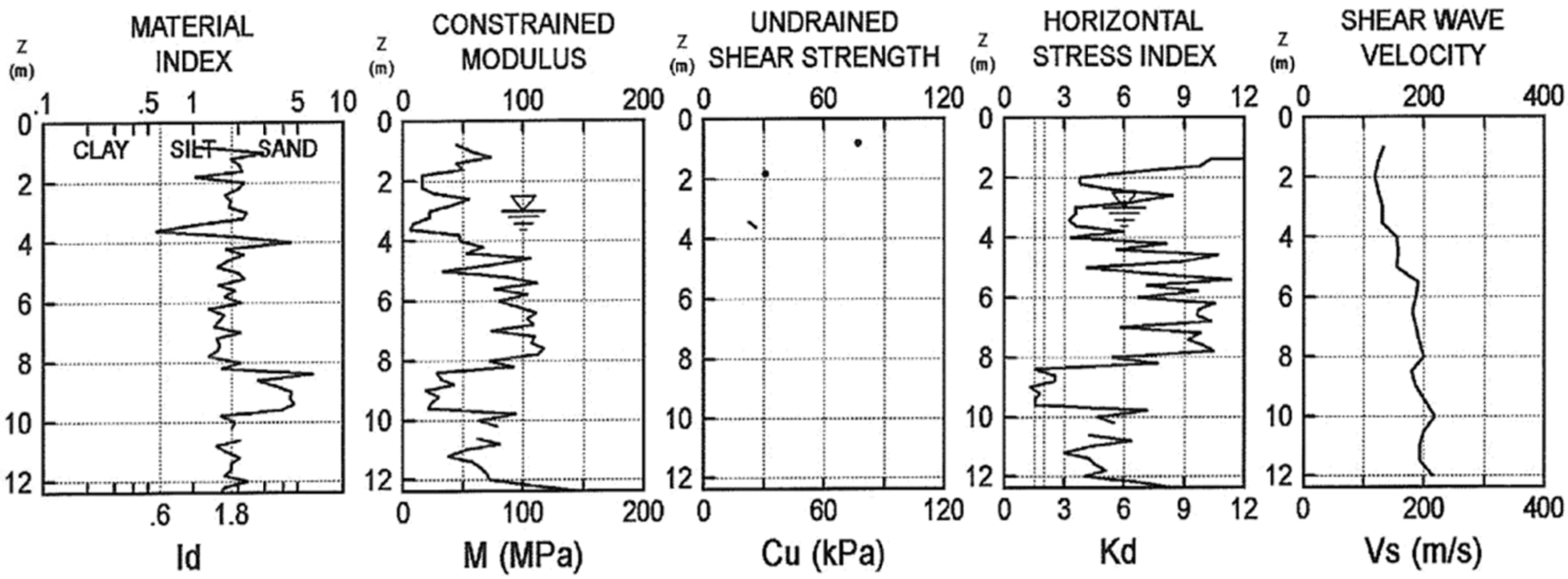
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X1250
INTERPRETED GEOTECHNICAL PARAMETERS		12 JAN 2016

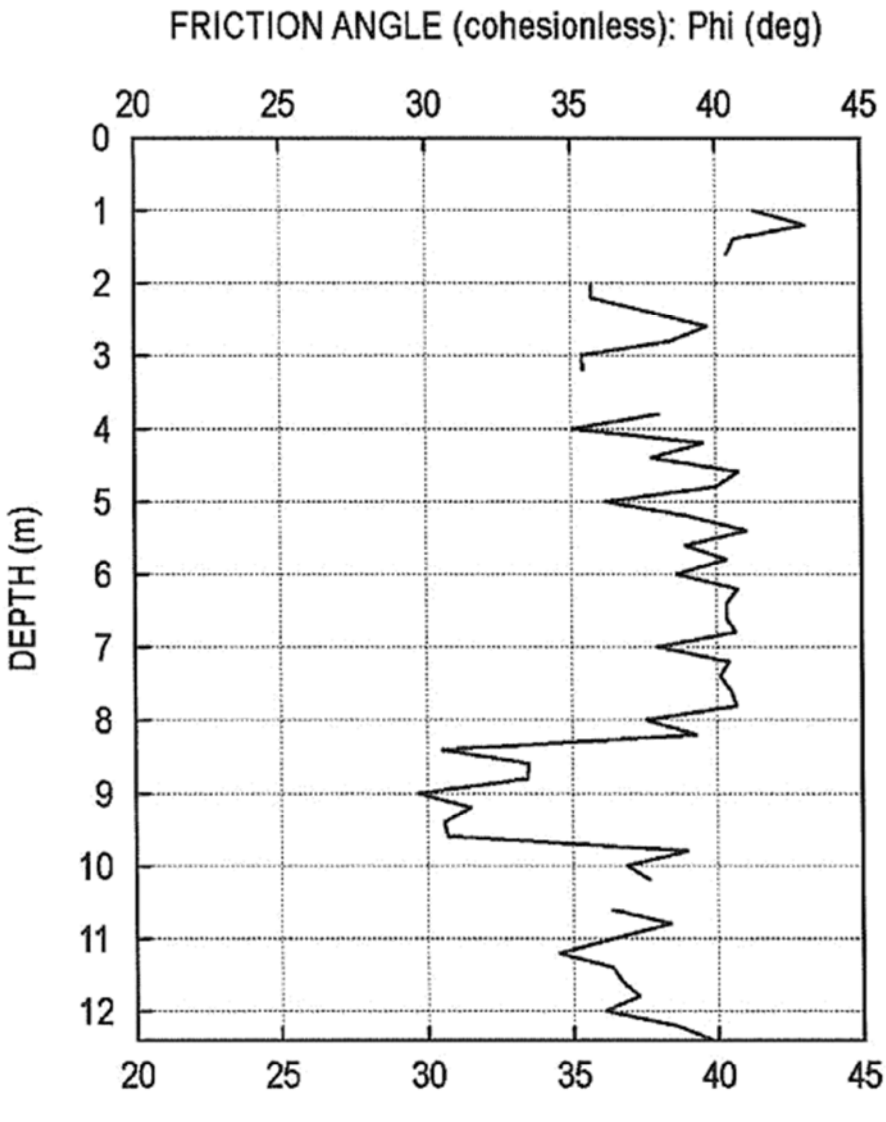


SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST SDMT-X1450 12 JAN 2016



Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X1450
INTERPRETED GEOTECHNICAL PARAMETERS		12 JAN 2016



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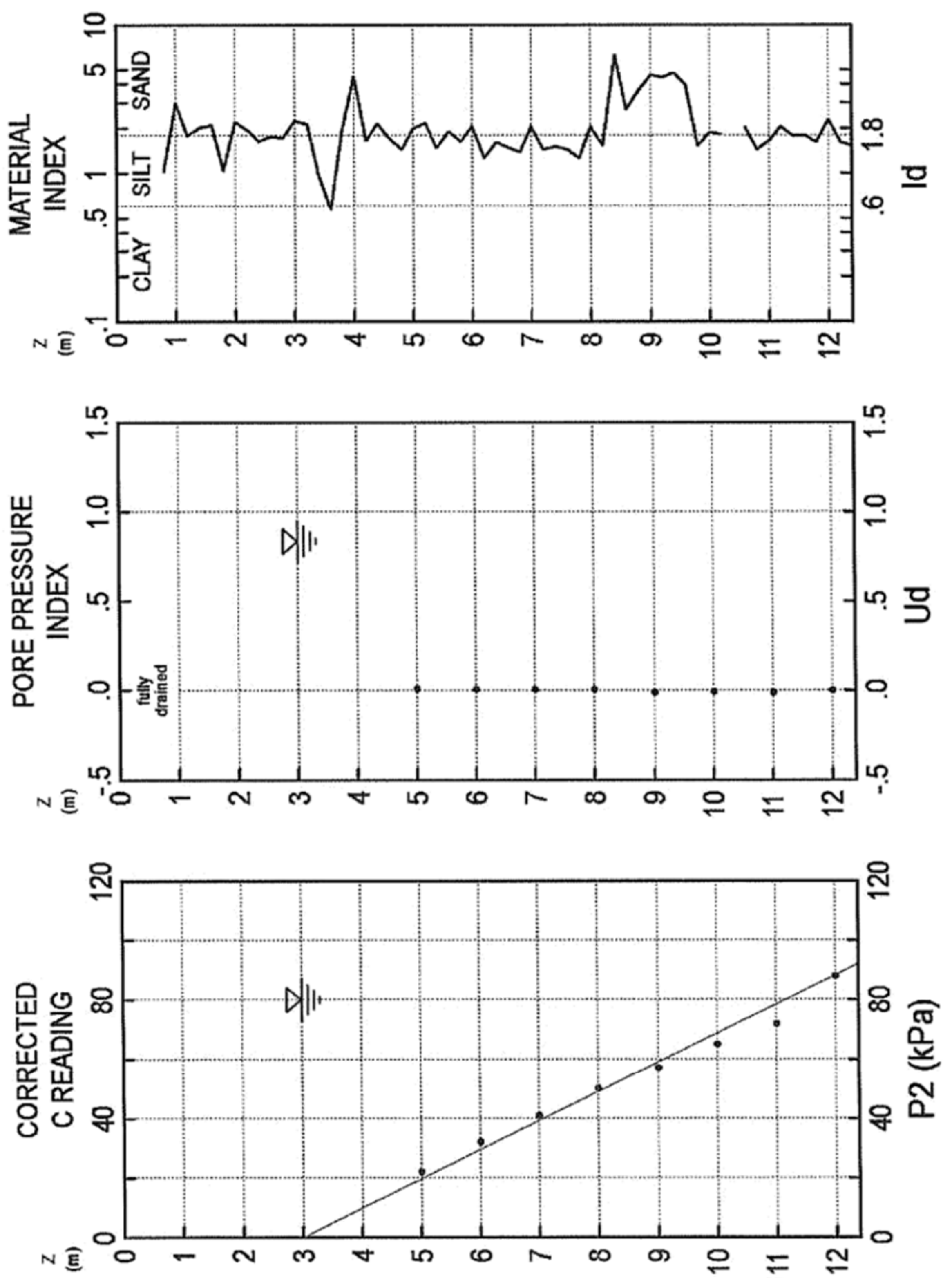
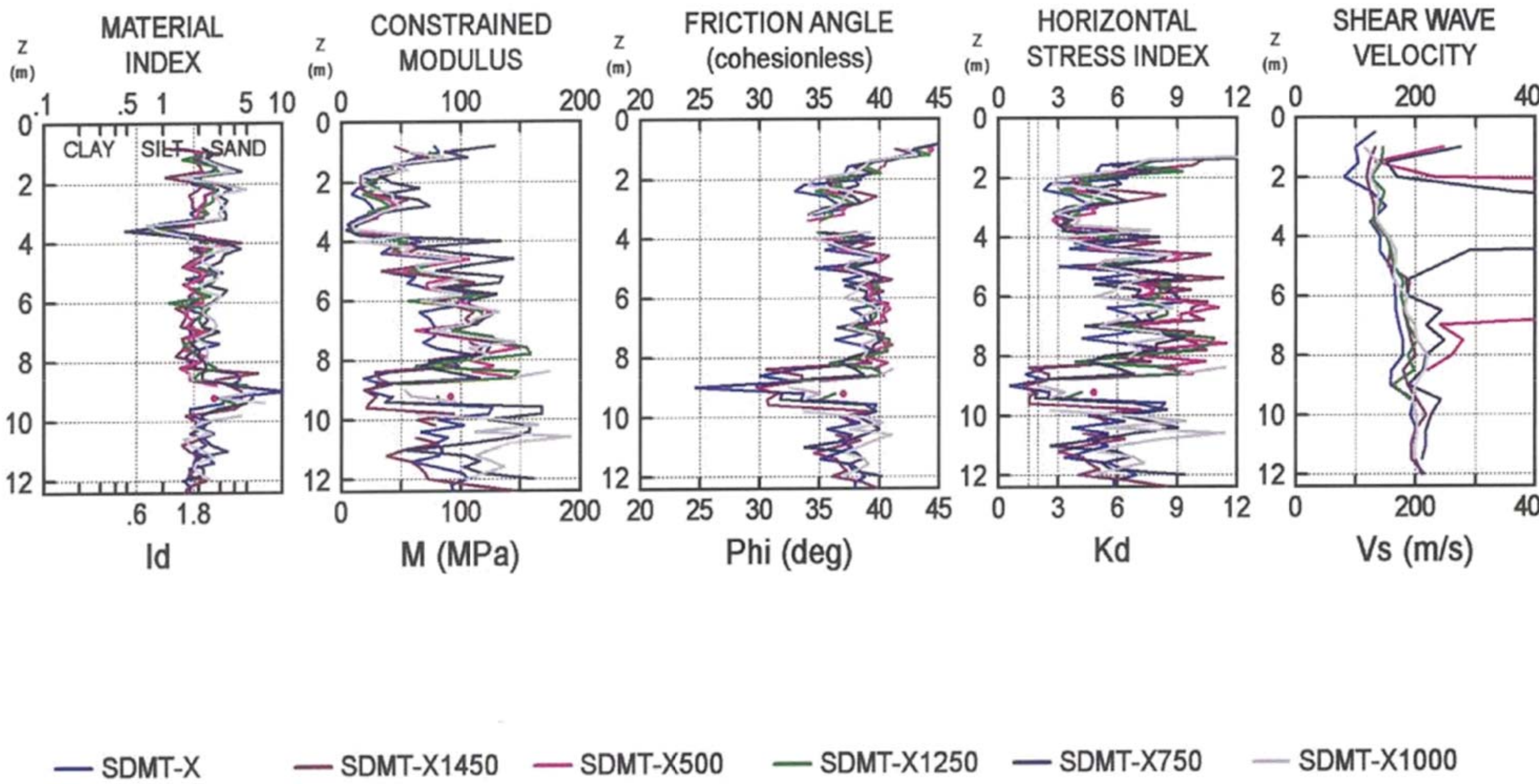
DILATOMETER TEST (DMT)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-X1450
INTERPRETED GEOTECHNICAL PARAMETERS		12 JAN 2016

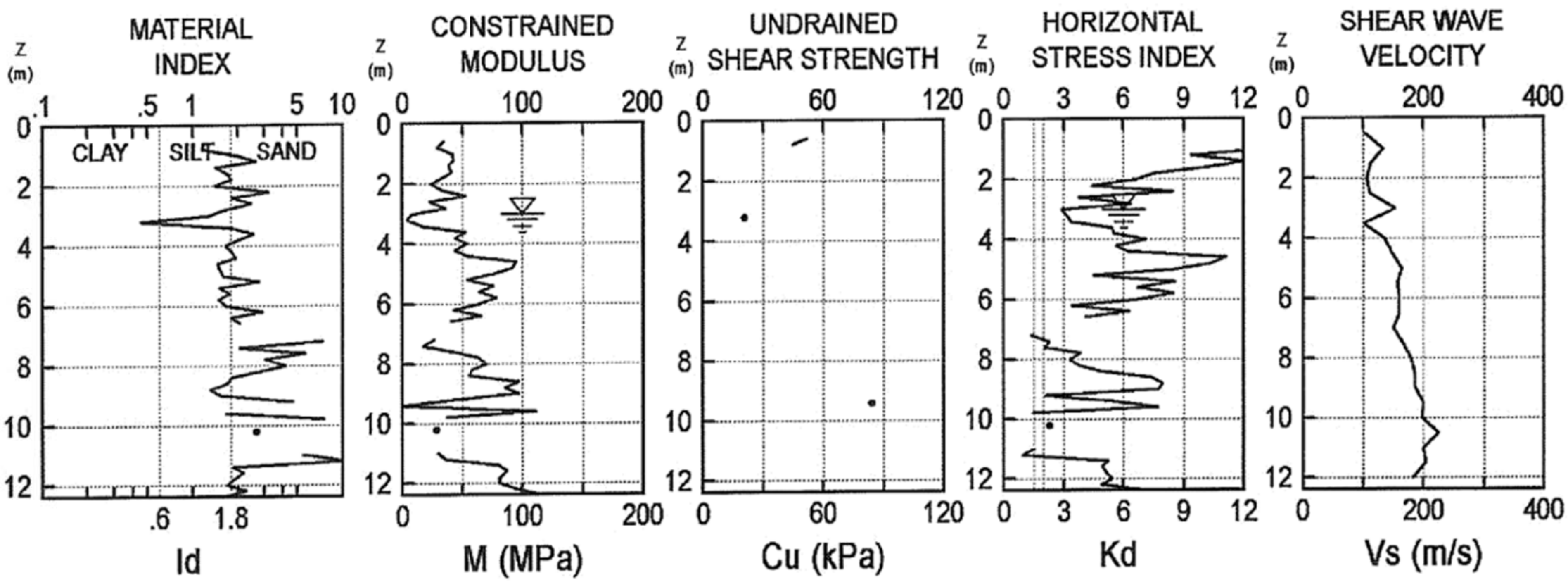


Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd



SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-Y** 10 SEP 2015



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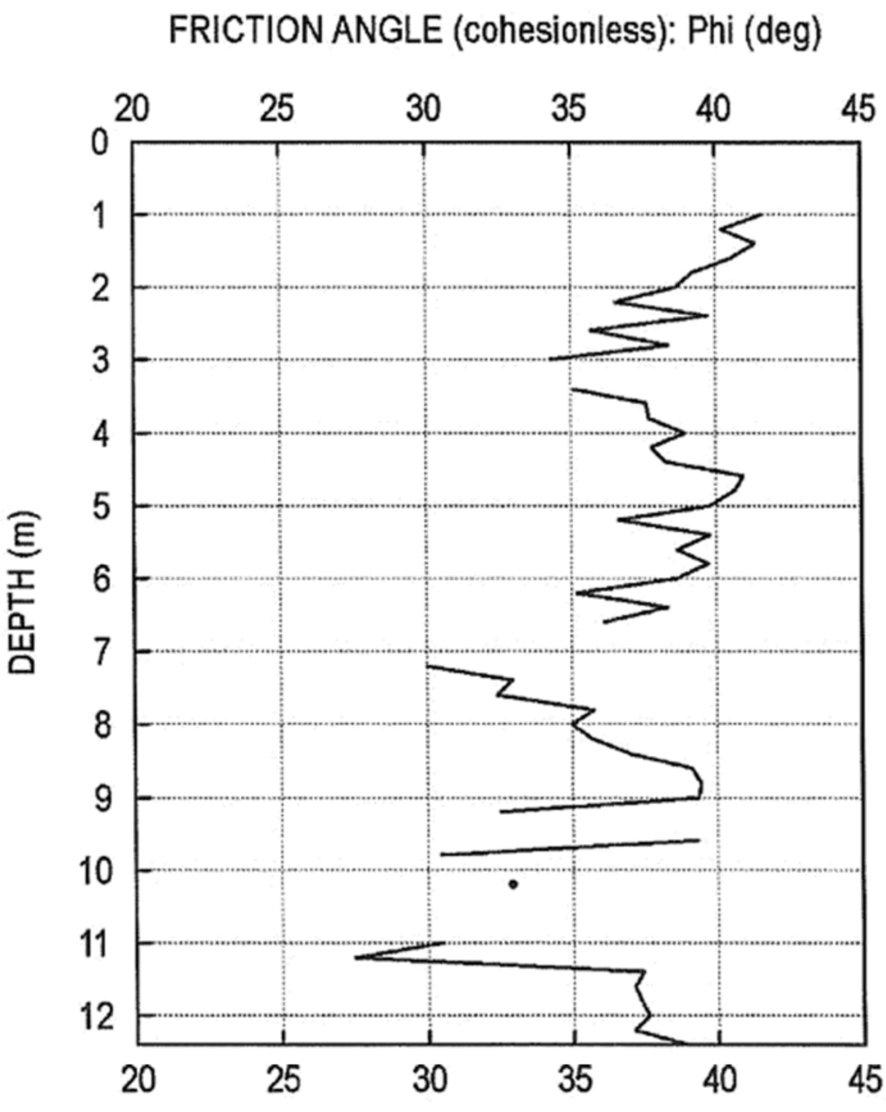
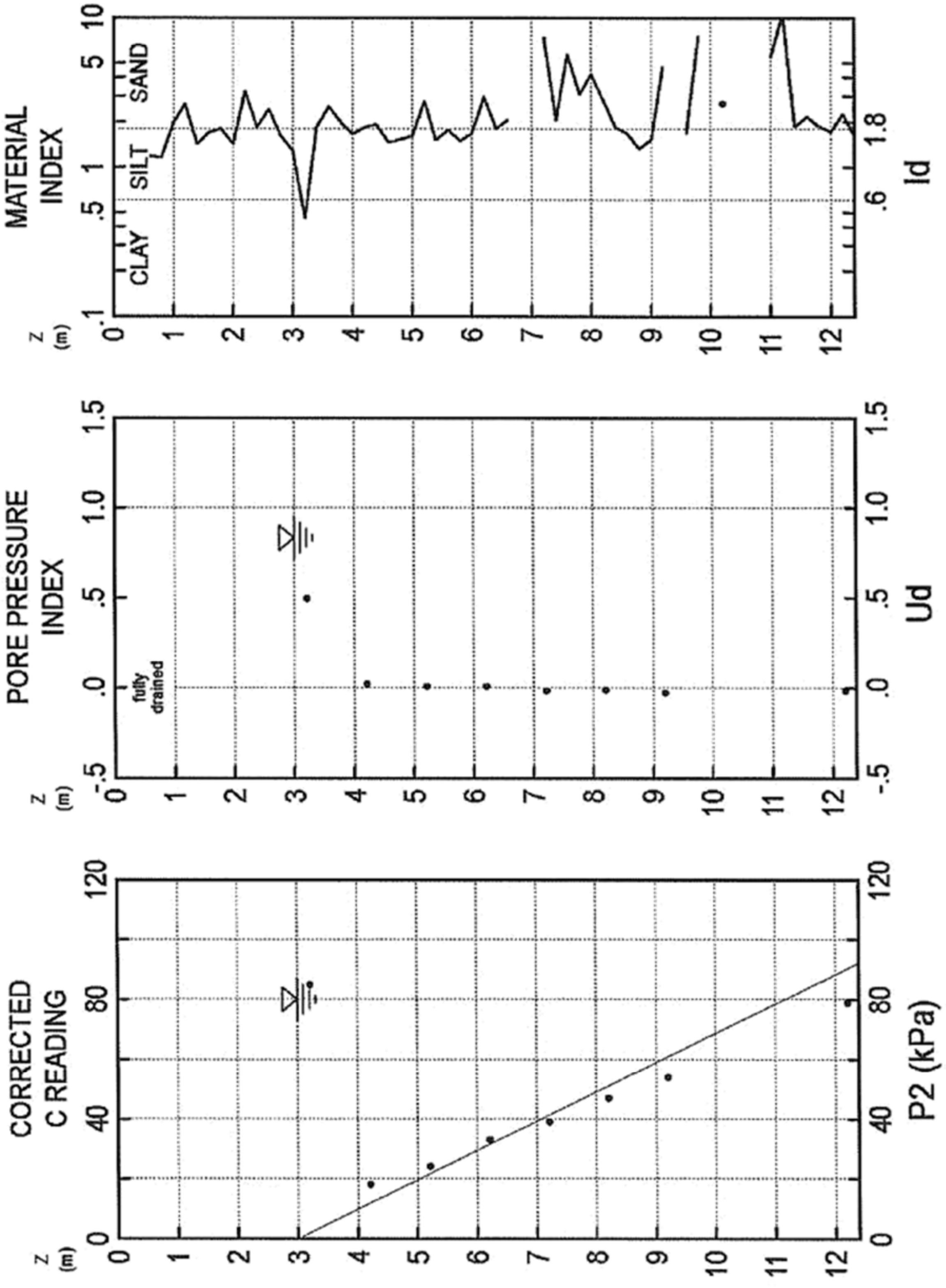
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-Y**
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



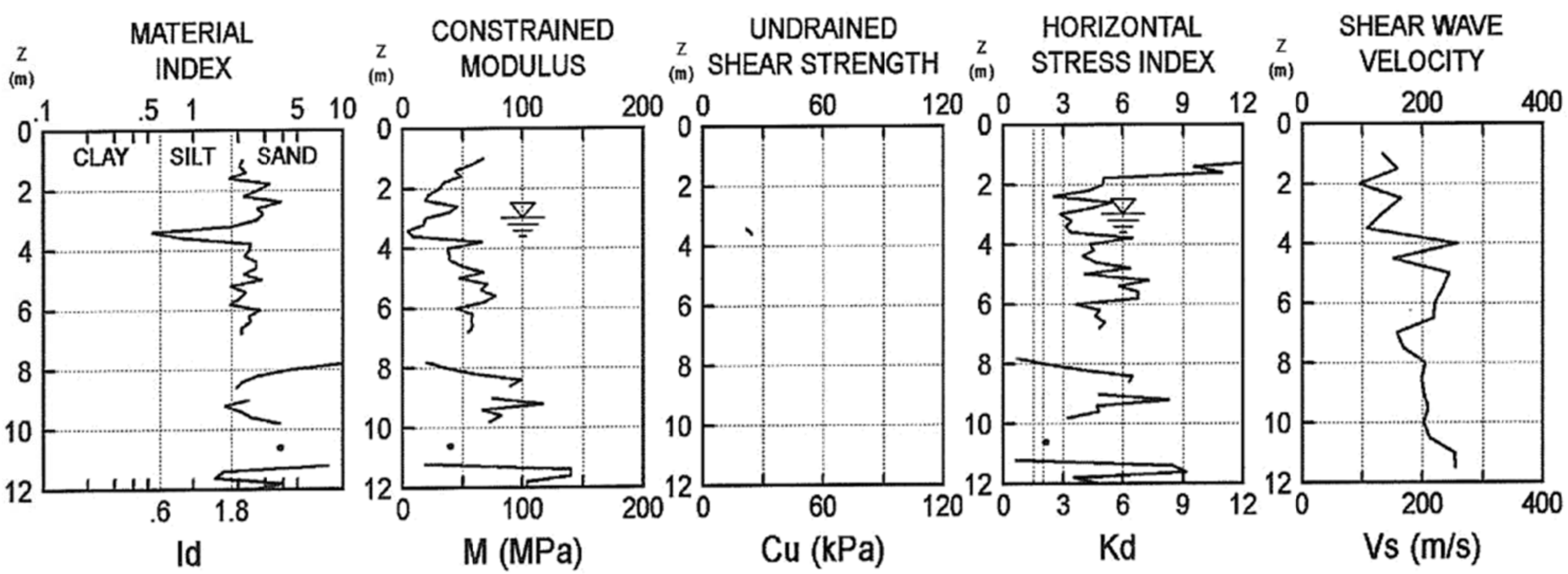
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST SDMT-Y500 15 JAN 2016



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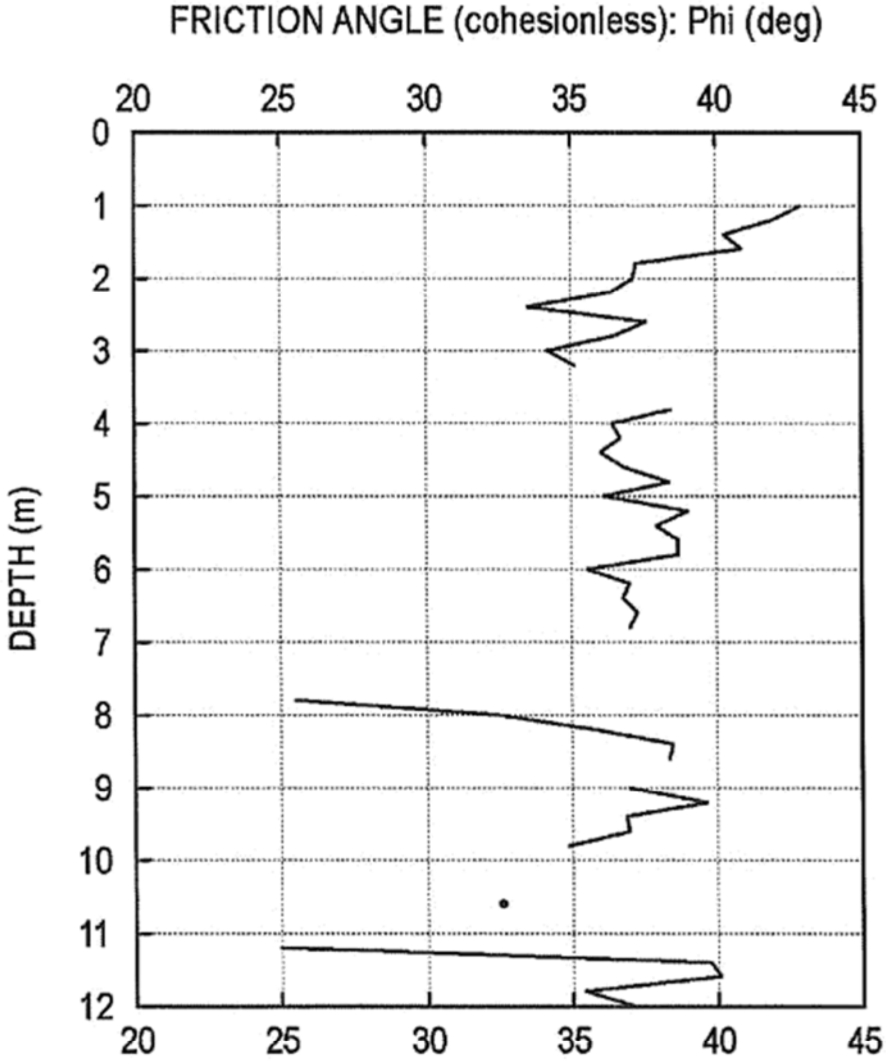
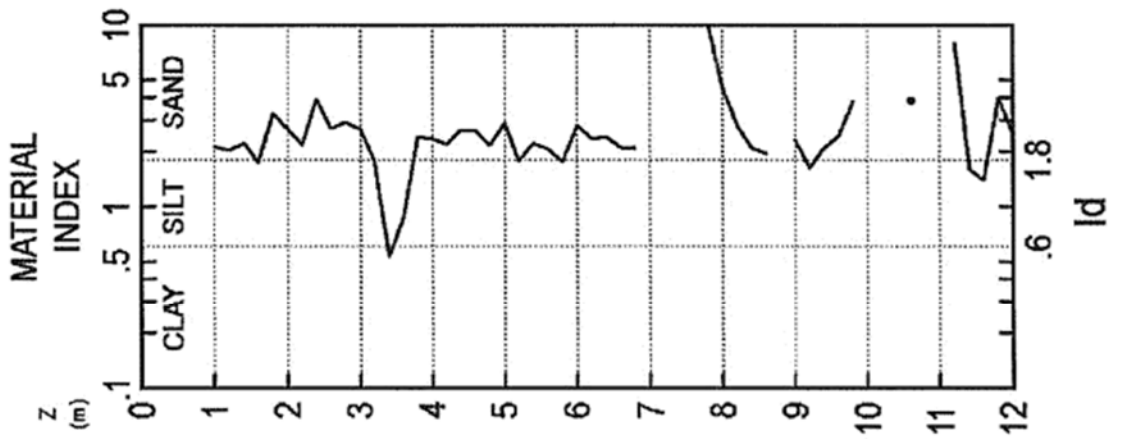
DILATOMETER TEST (D M T)

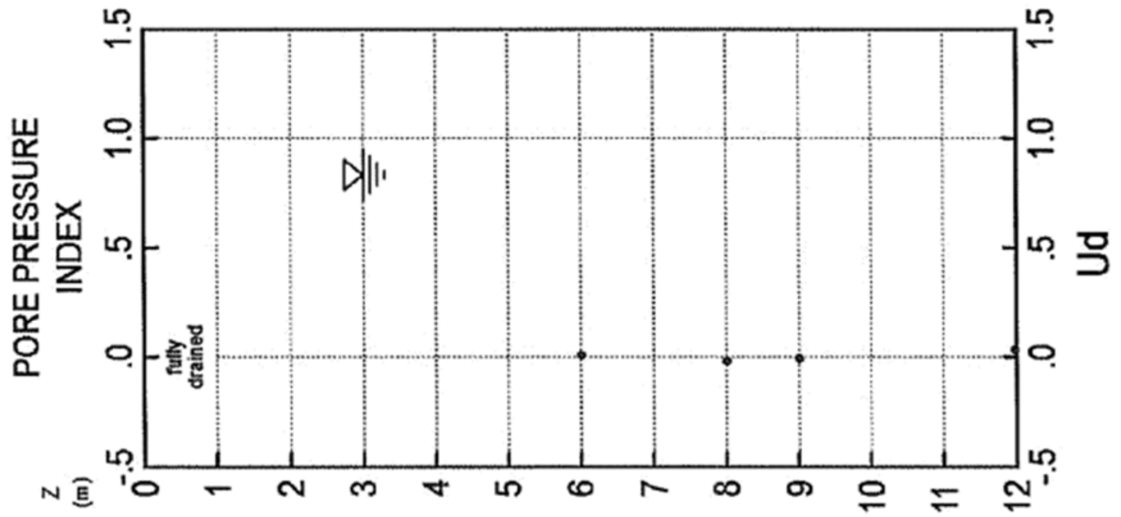
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-Y500**
INTERPRETED GEOTECHNICAL PARAMETERS		15 JAN 2016

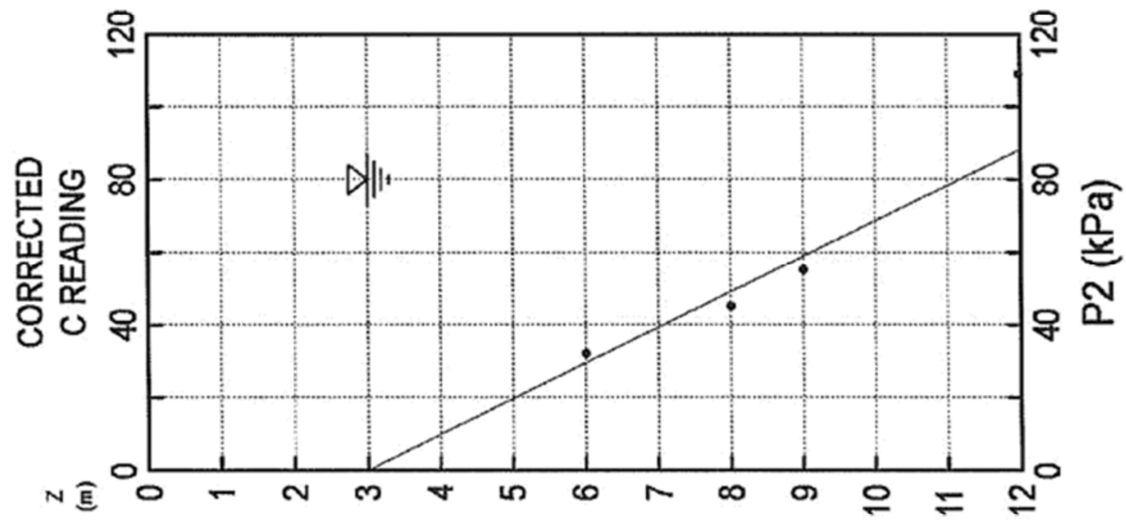


DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y500
INTERPRETED GEOTECHNICAL PARAMETERS		15 JAN 2016

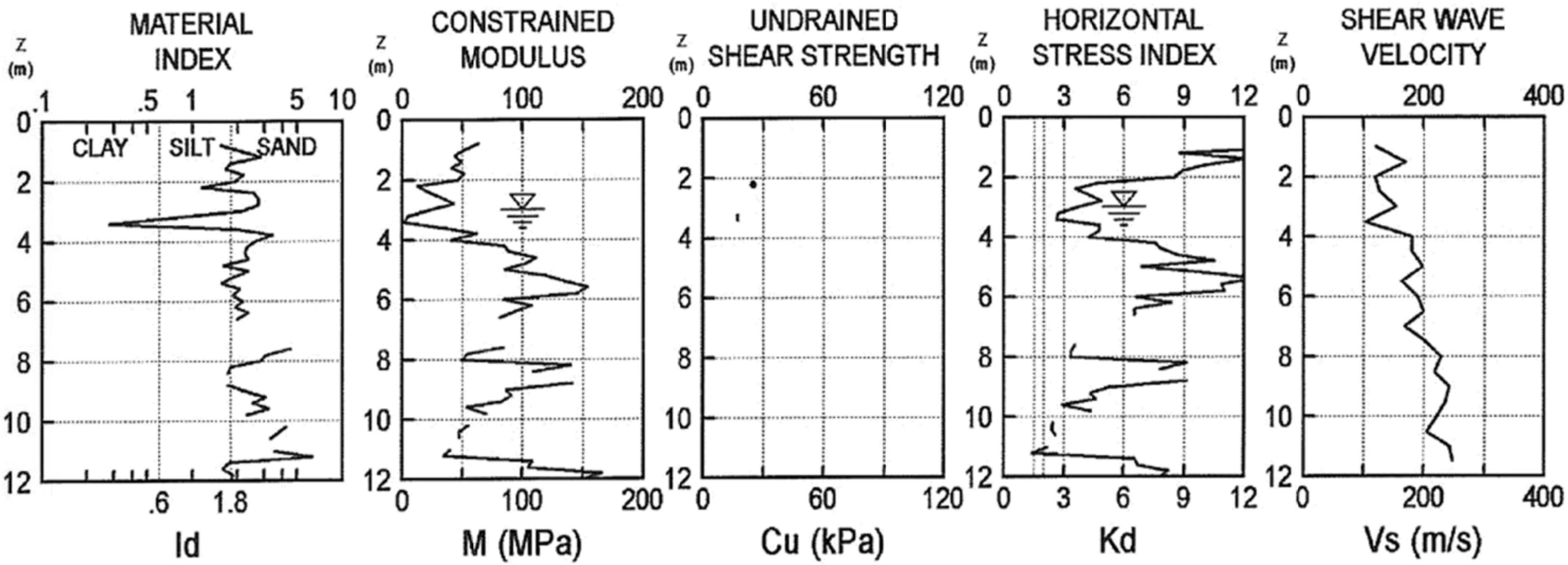




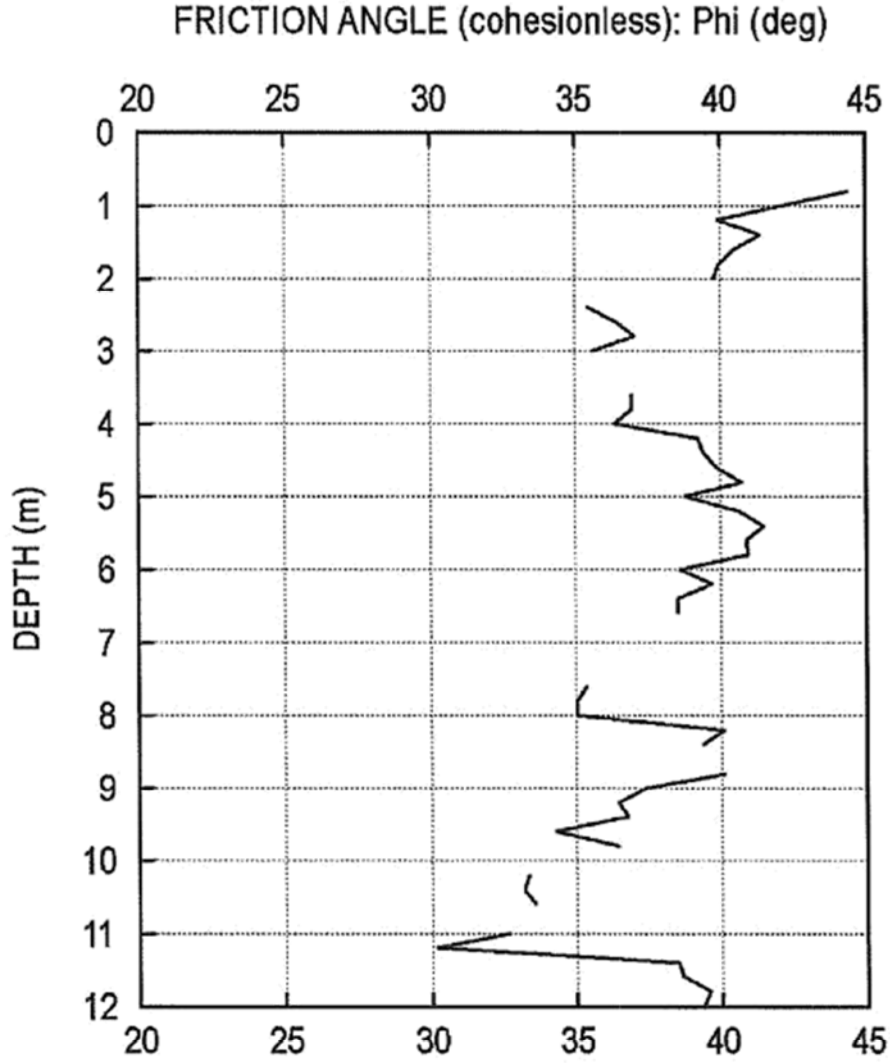


SEISMIC DILATOMETER TEST (S D M T)

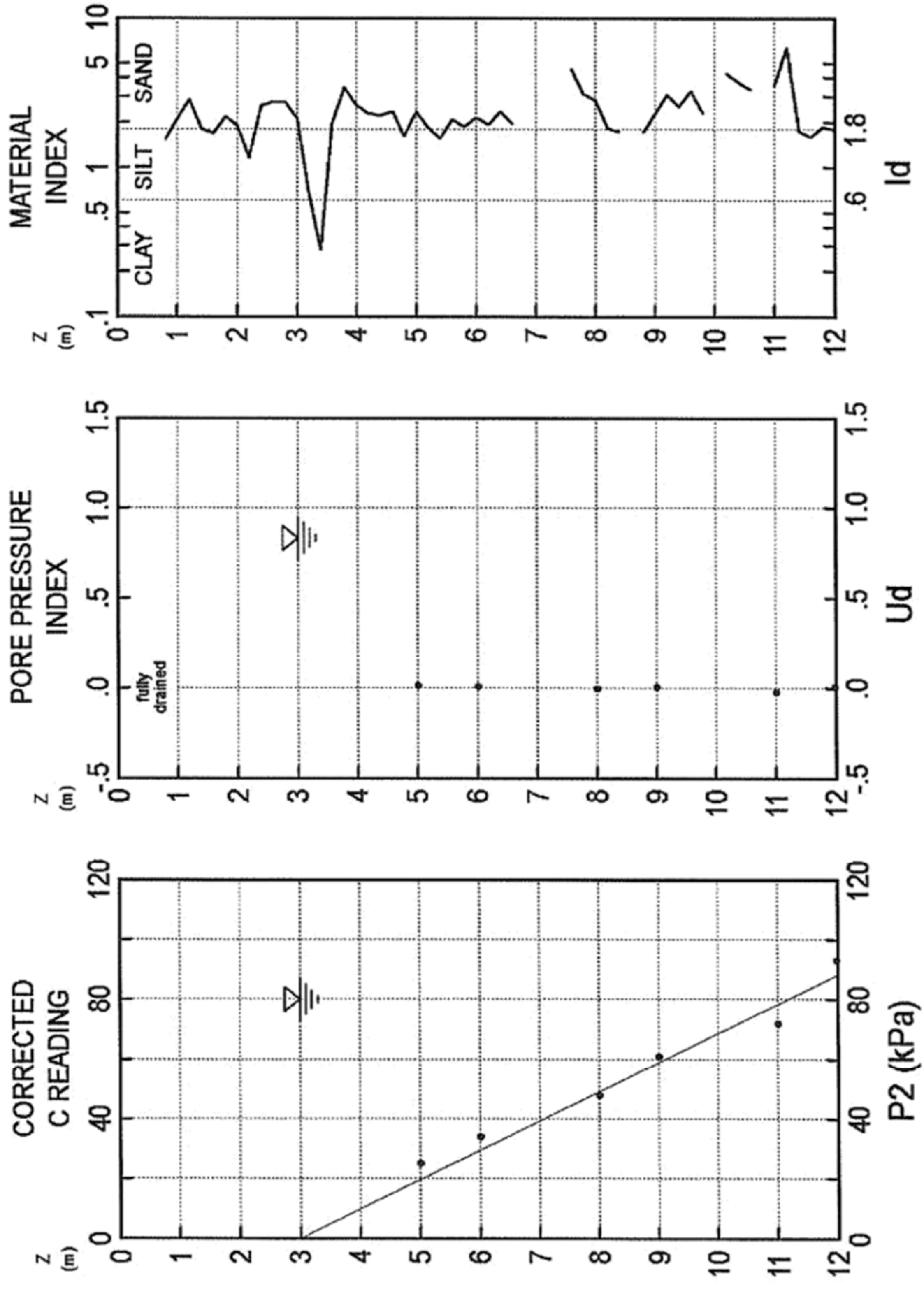
Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-Y750** 13 JAN 2016



Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y750
INTERPRETED GEOTECHNICAL PARAMETERS		13 JAN 2016

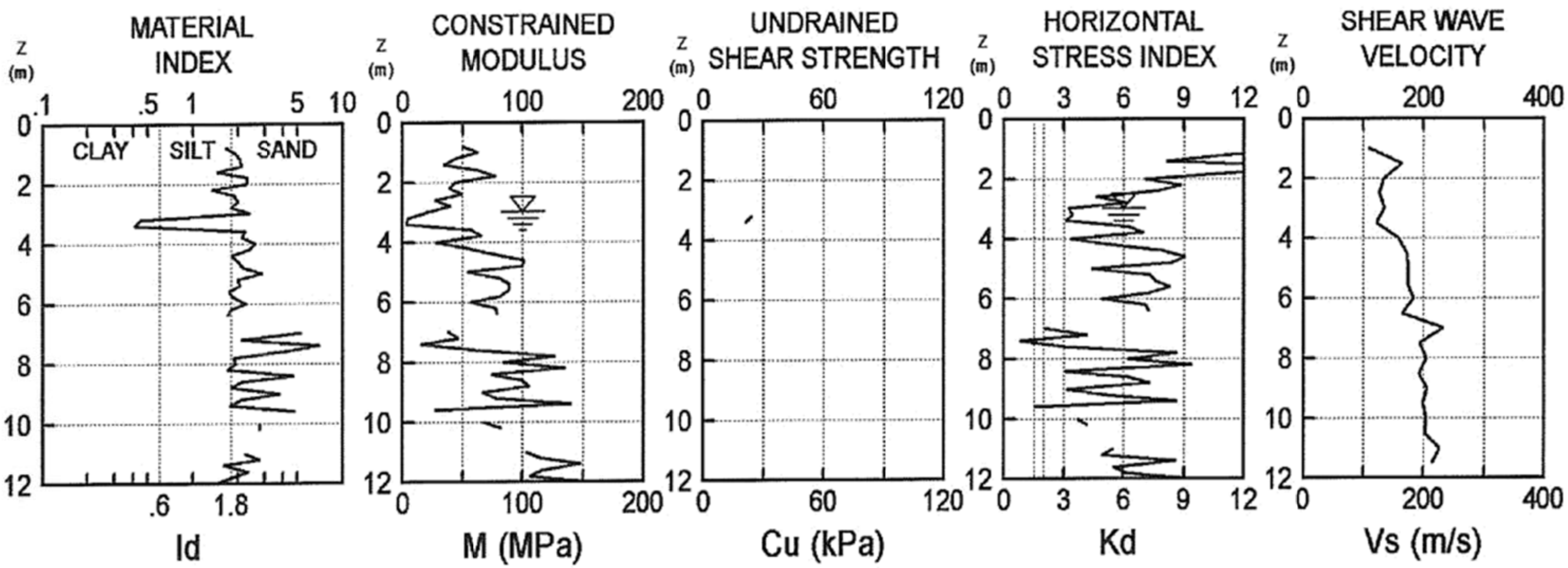


Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y750
INTERPRETED GEOTECHNICAL PARAMETERS		13 JAN 2016



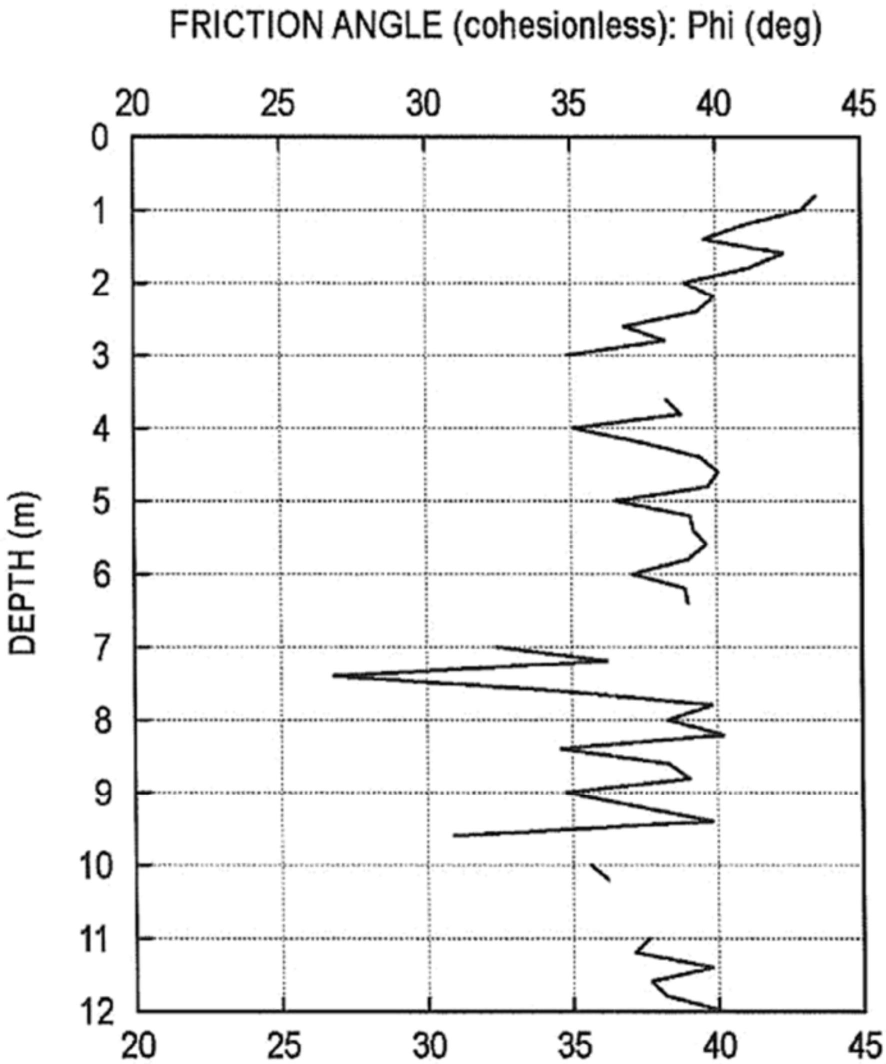
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST SDMT-Y1000 13 JAN 2016



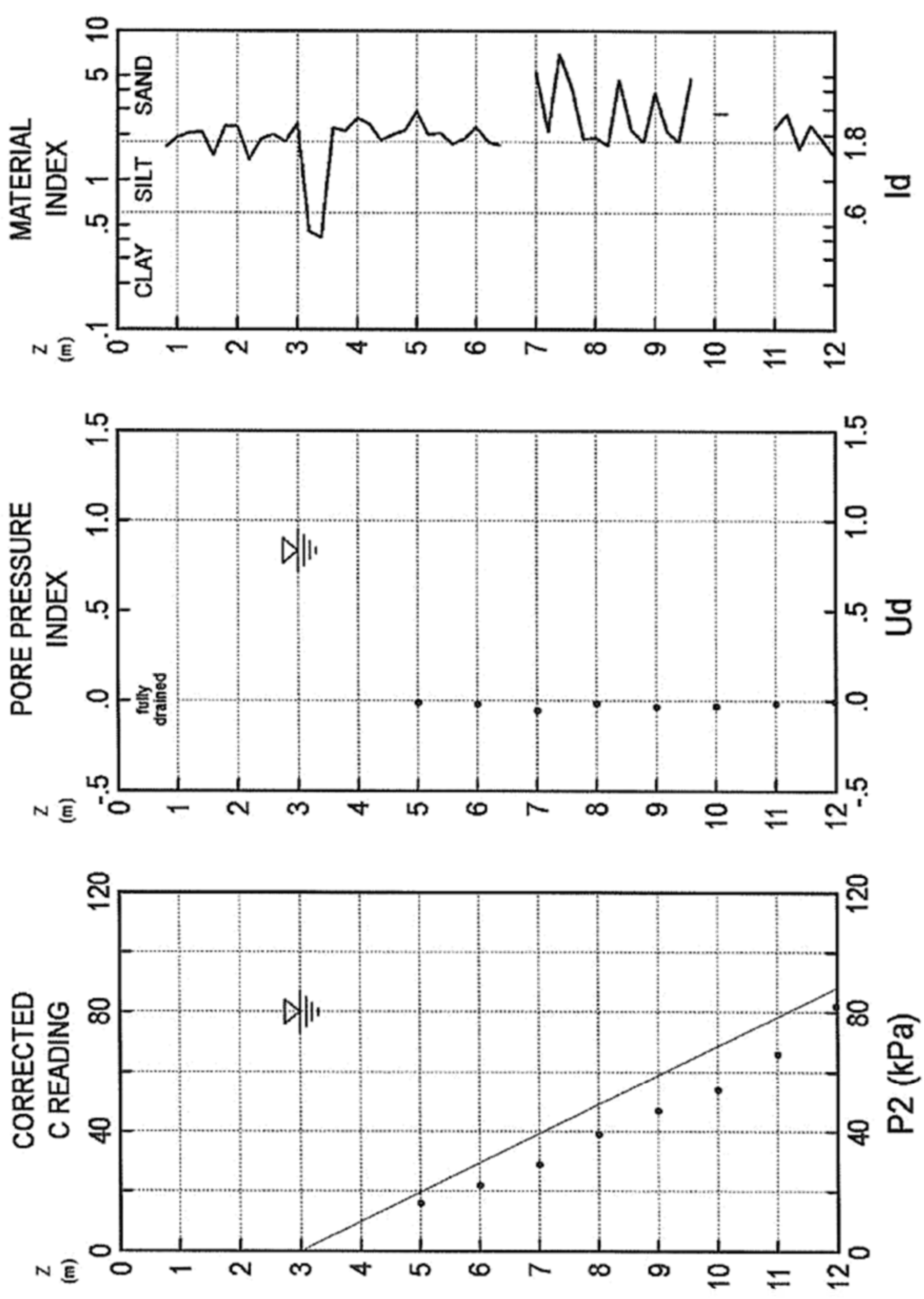
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y1000
INTERPRETED GEOTECHNICAL PARAMETERS		13 JAN 2016



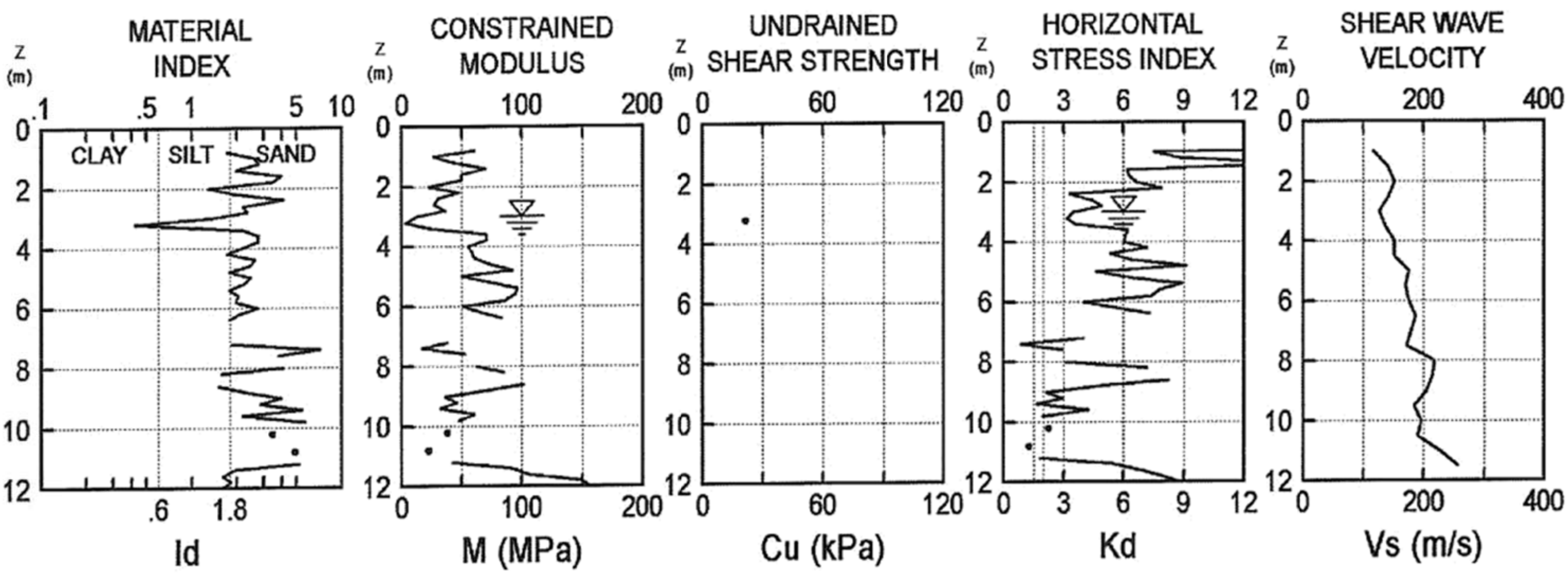
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y1000
INTERPRETED GEOTECHNICAL PARAMETERS		13 JAN 2016



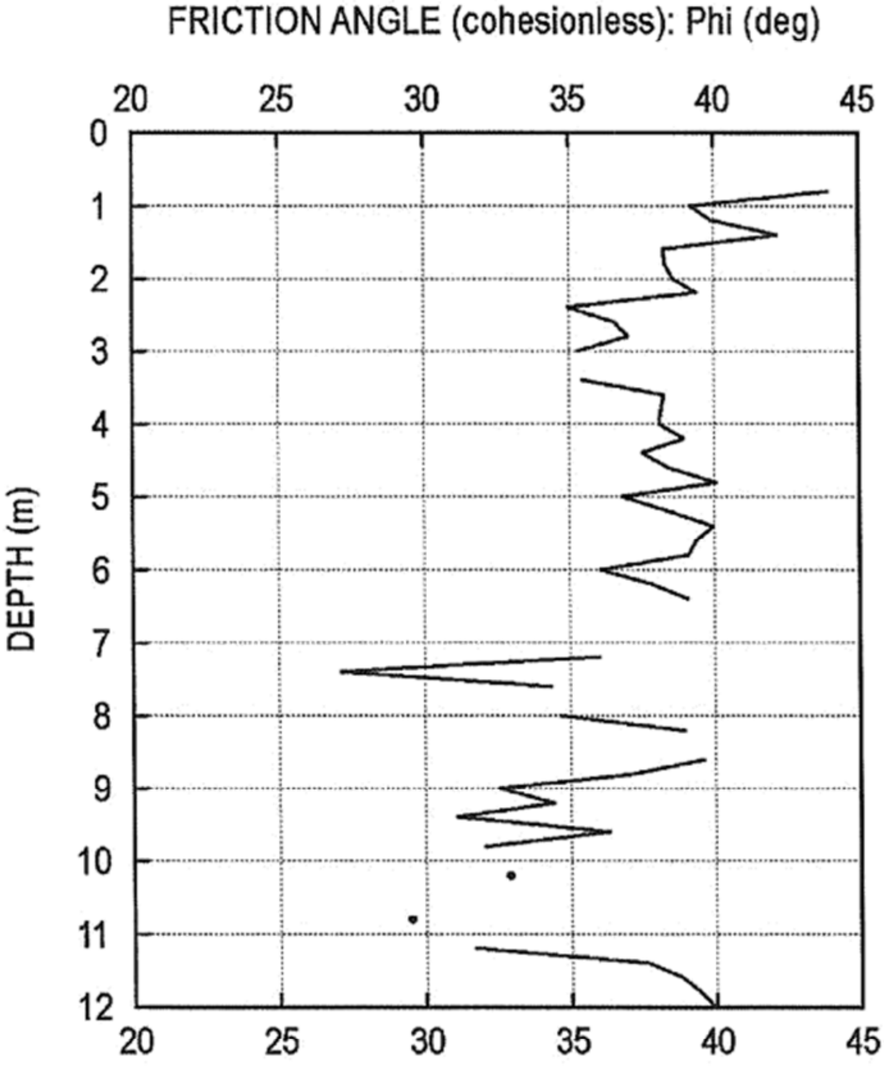
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-Y1250** 14 JAN 2016



DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y1250
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016



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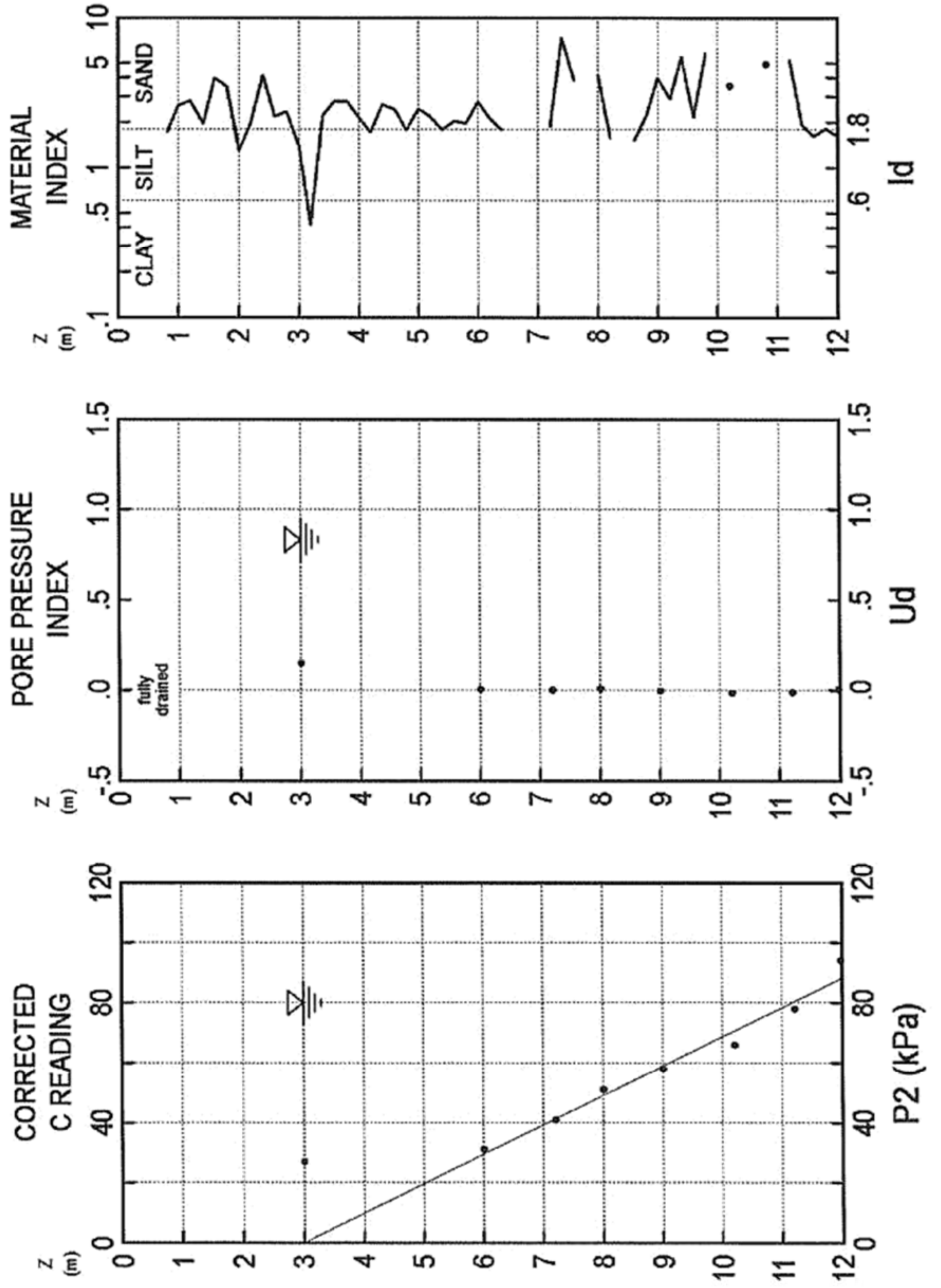
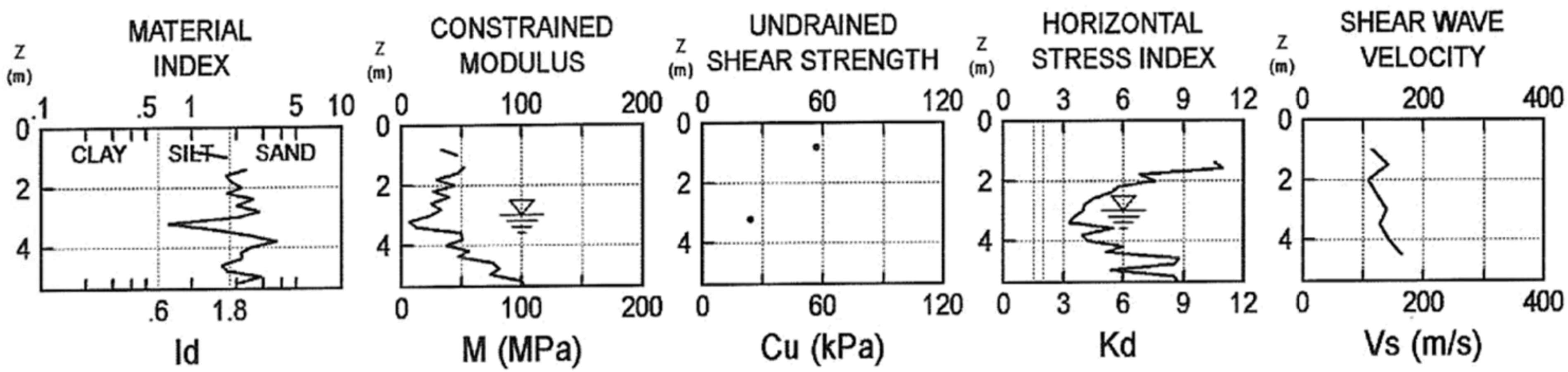
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-Y1250**
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016



SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-Y1450** 14 JAN 2016



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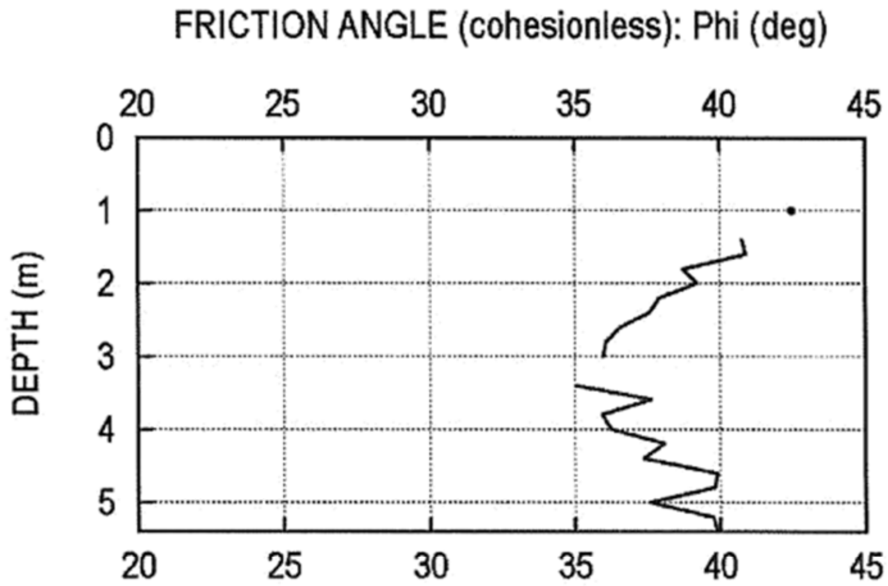
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y1450
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016



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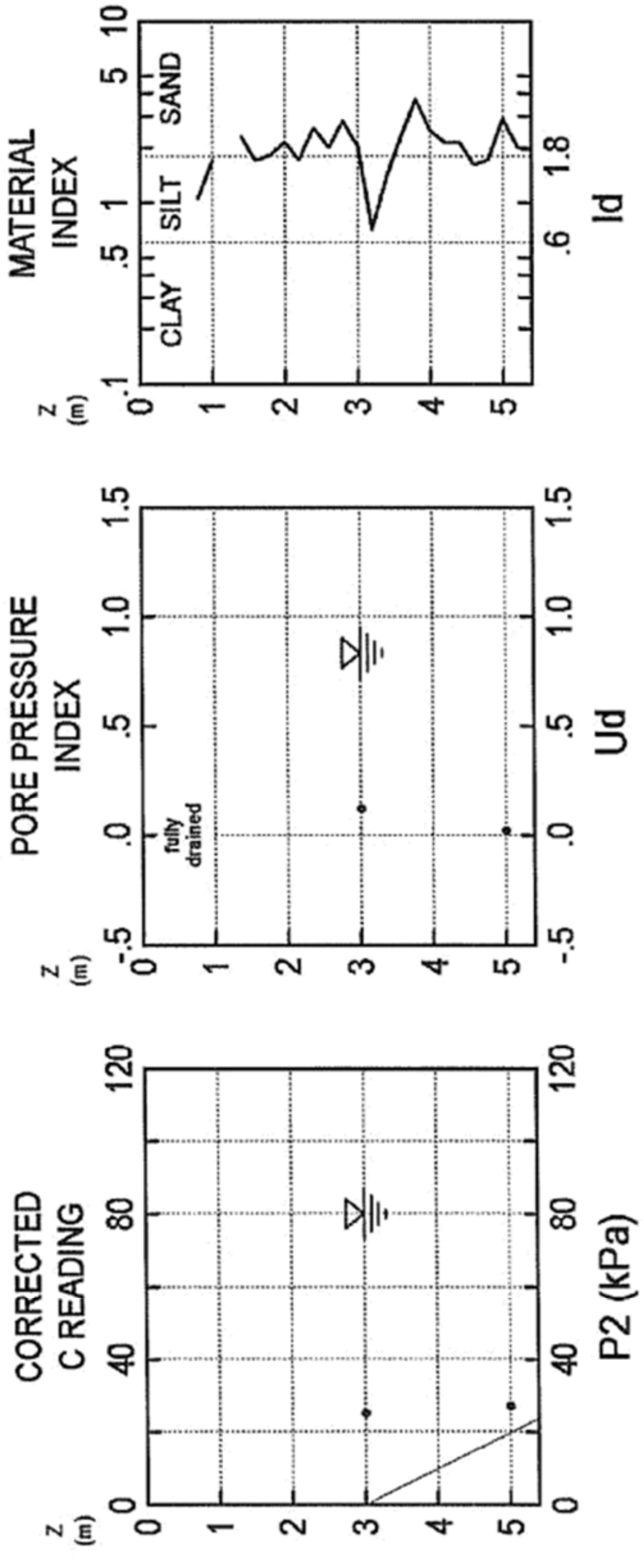
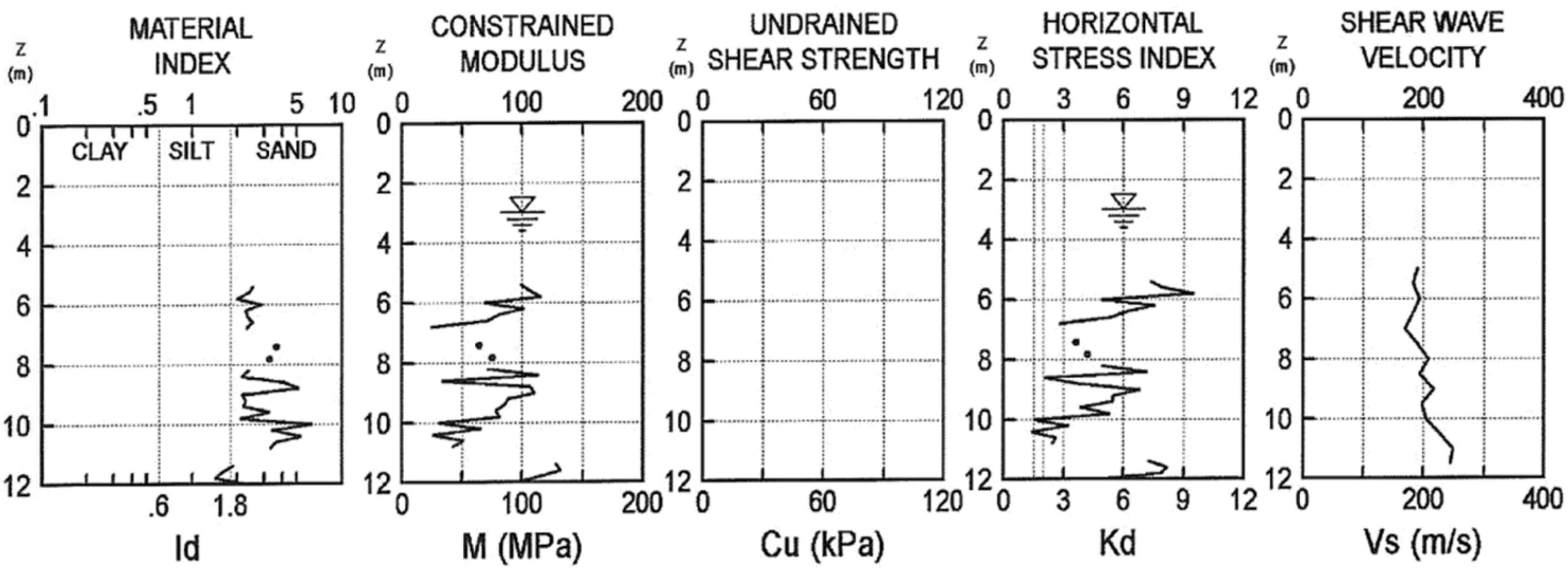
DILATOMETER TEST (DMT)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y1450
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016



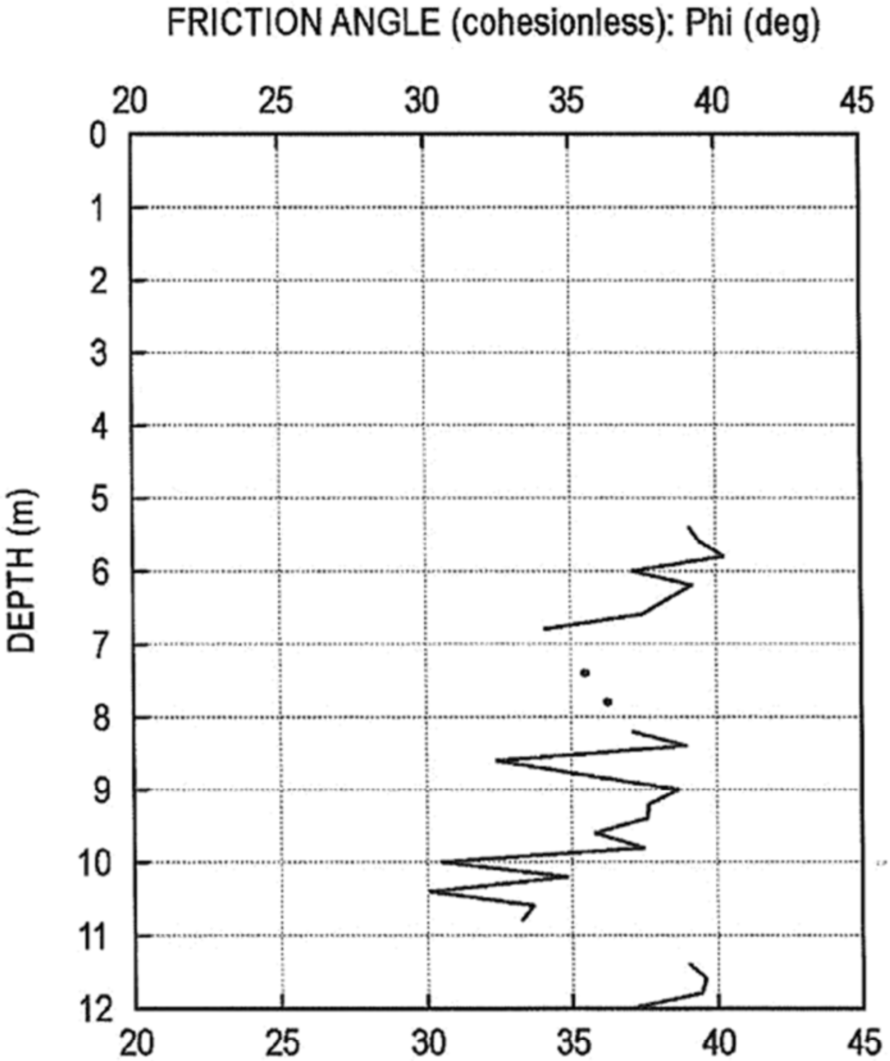
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-Y1450a** 14 JAN 2016



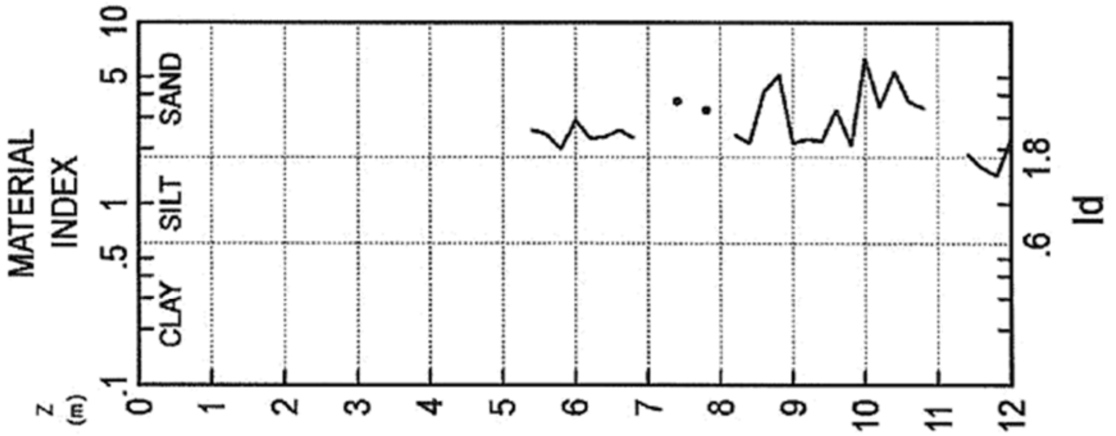
DILATOMETER TEST (D M T)

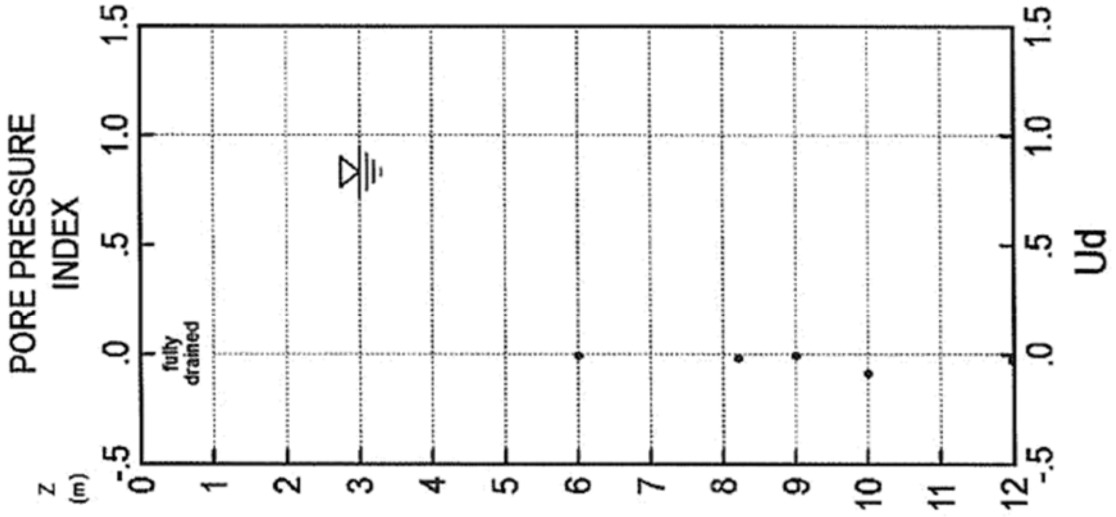
Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Y1450a
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016

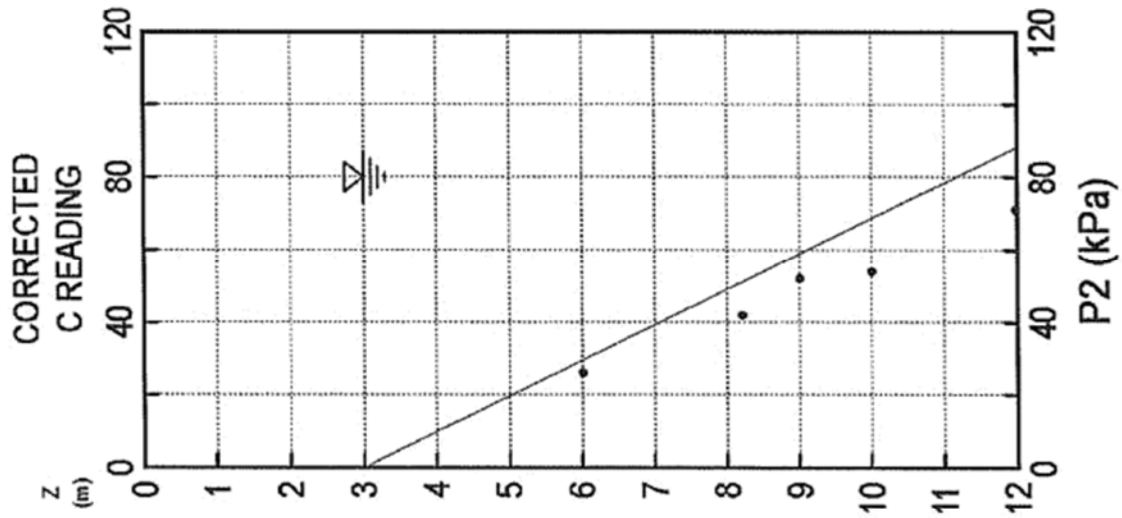


DILATOMETER TEST (DMT)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-Y1450a**
INTERPRETED GEOTECHNICAL PARAMETERS		14 JAN 2016

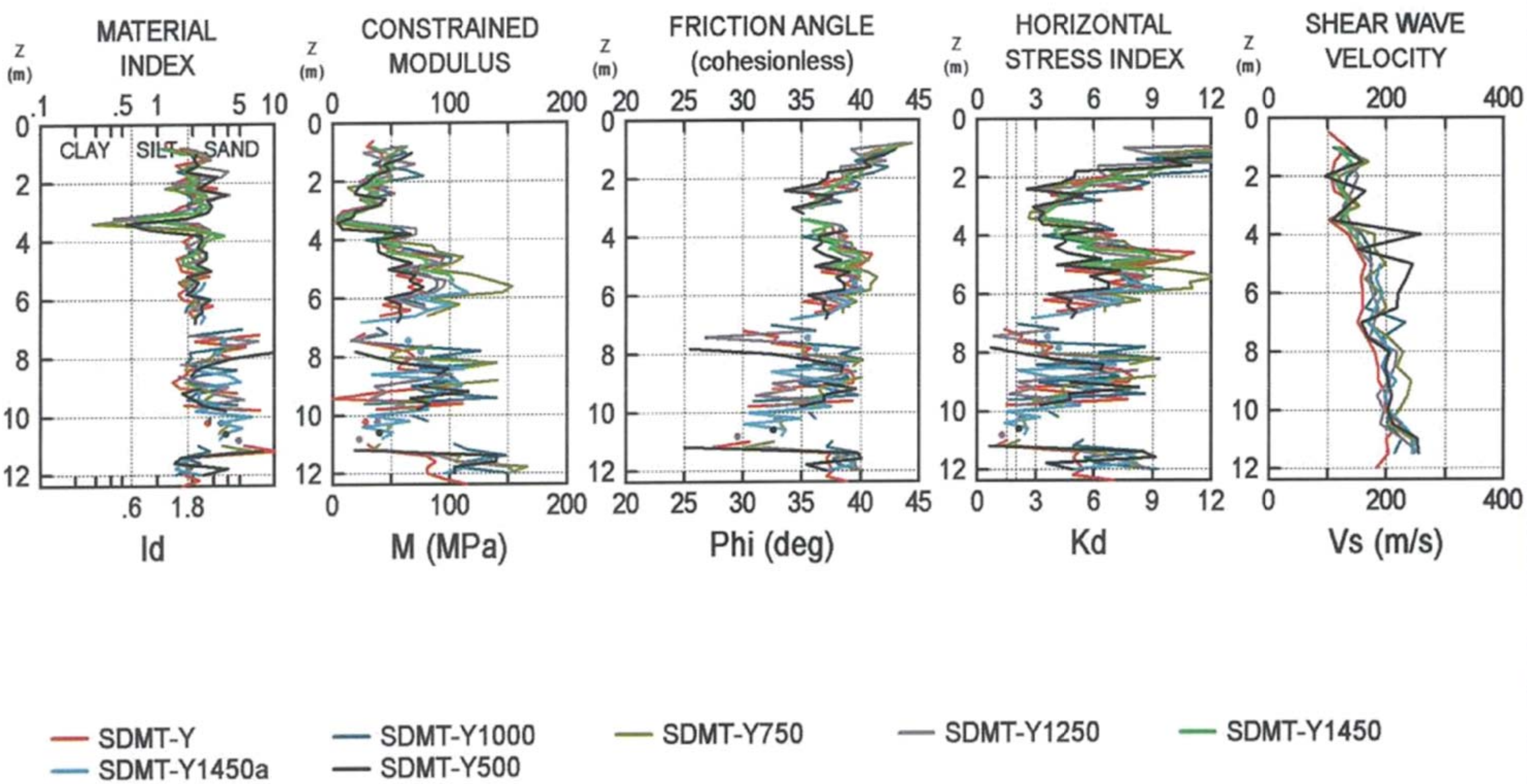






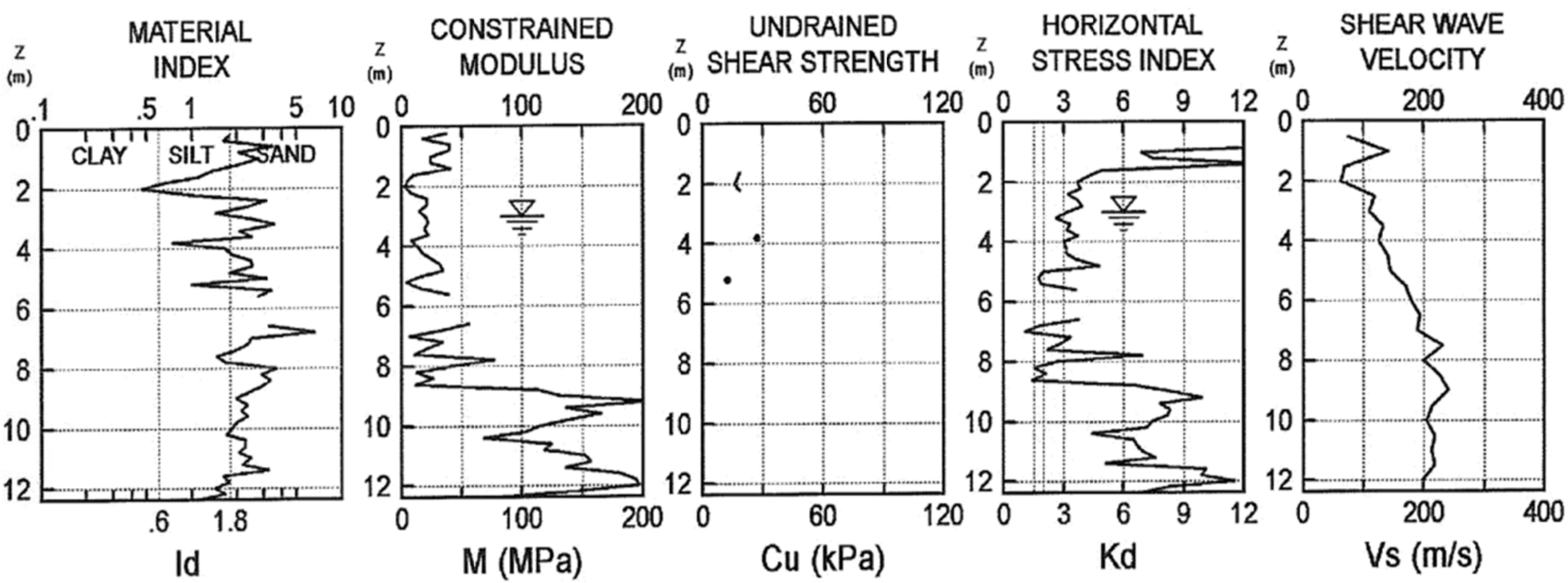
Ground Investigation
15-169

Hiways Geotechnical
27 Shirley Rd



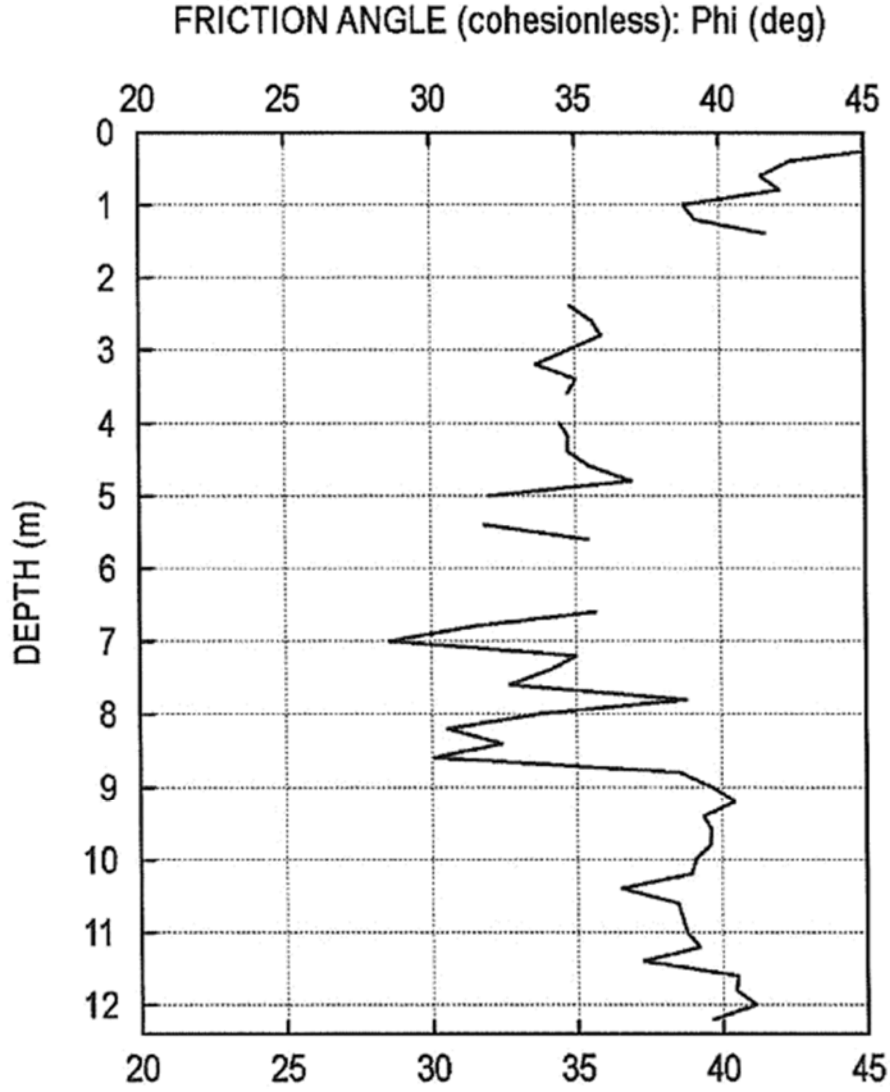
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation 15-169	Hiways Geotechnical 27 Shirley Rd	TEST **SDMT-Z** 10 SEP 2015



DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	SDMT-Z
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



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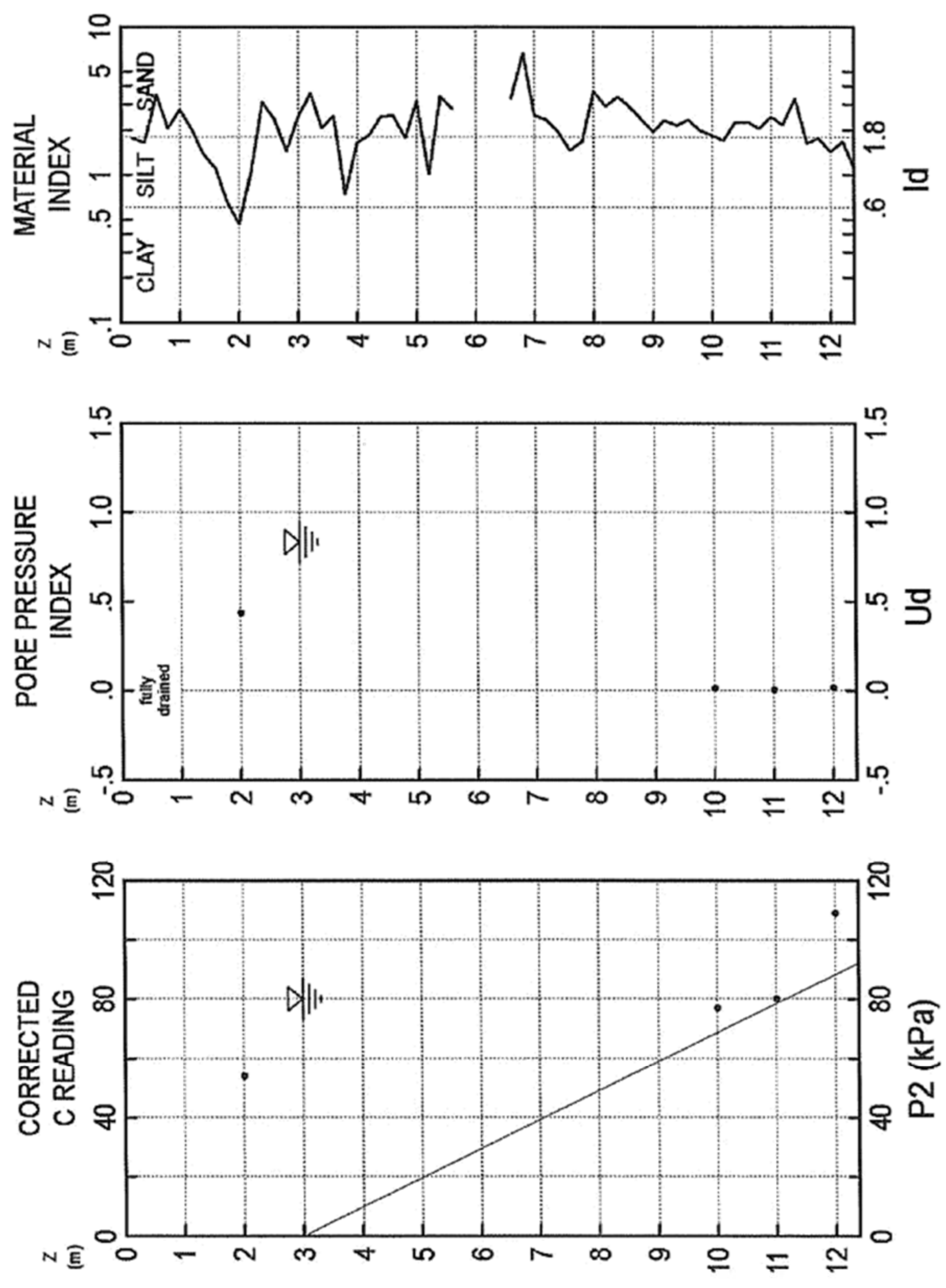
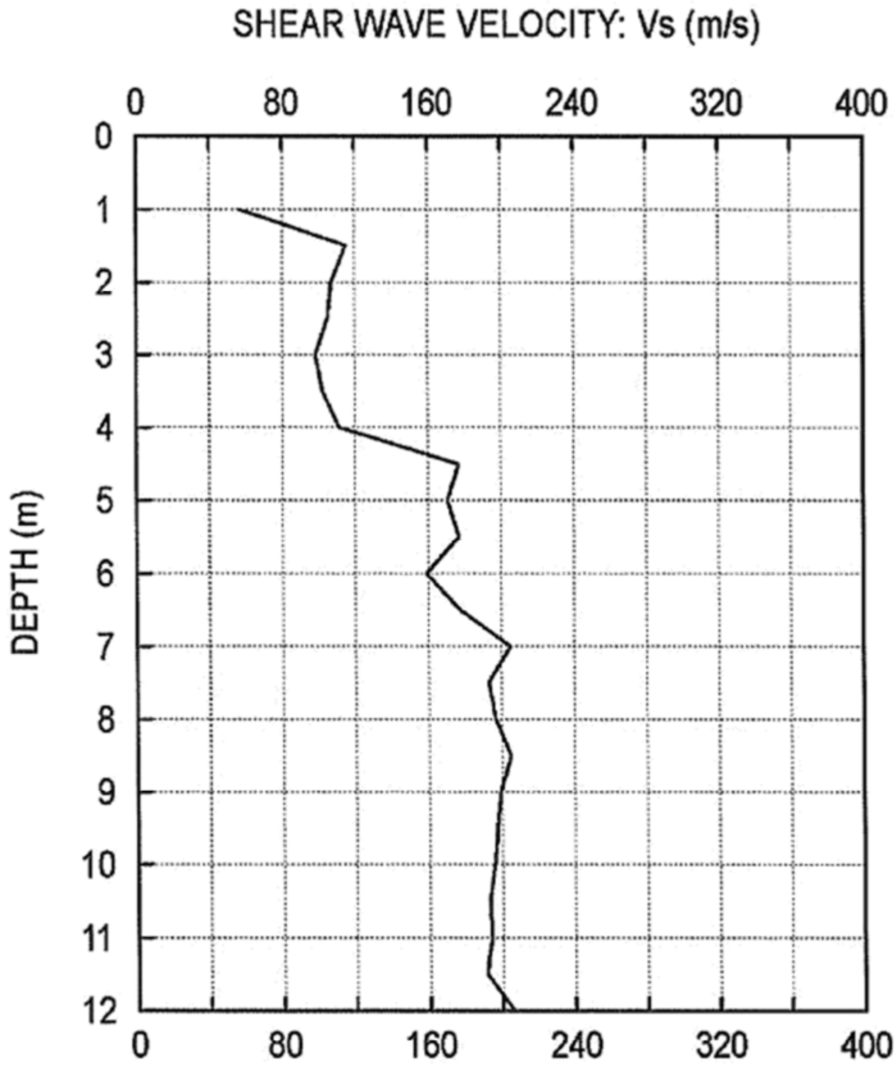
DILATOMETER TEST (D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**SDMT-Z**
INTERPRETED GEOTECHNICAL PARAMETERS		10 SEP 2015



SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	VS-Sec Pre
		10 SEP 2015



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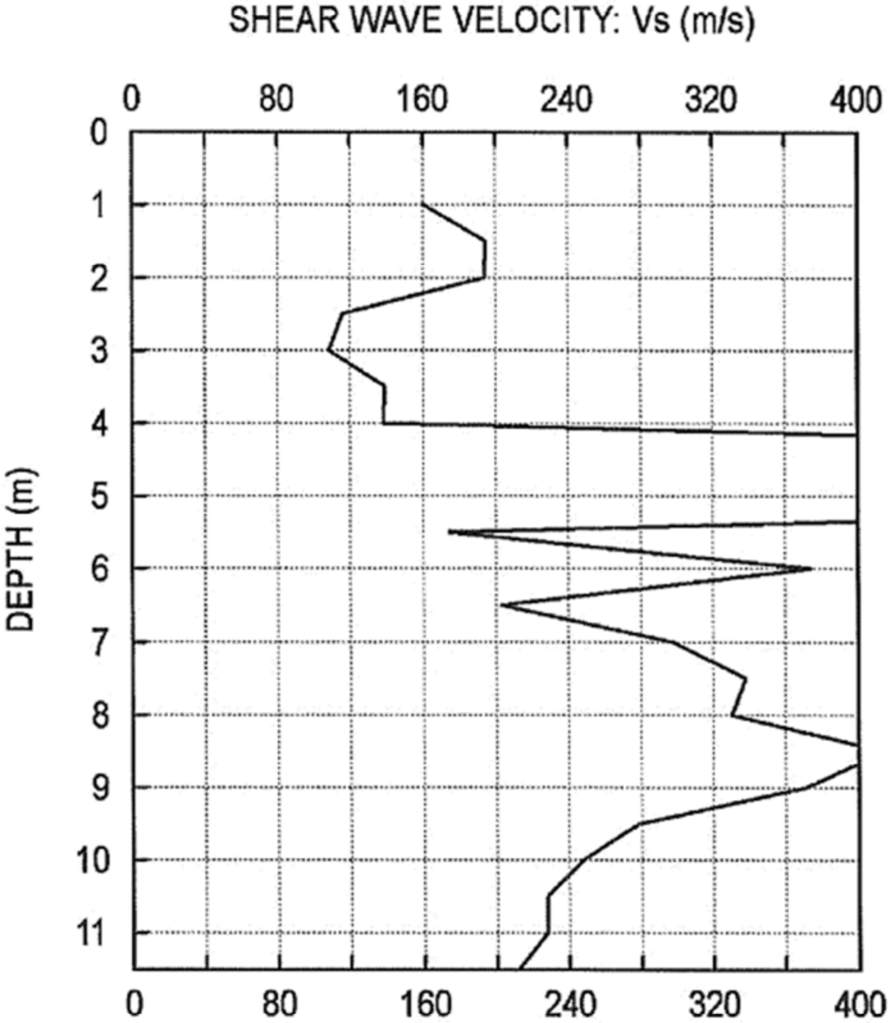
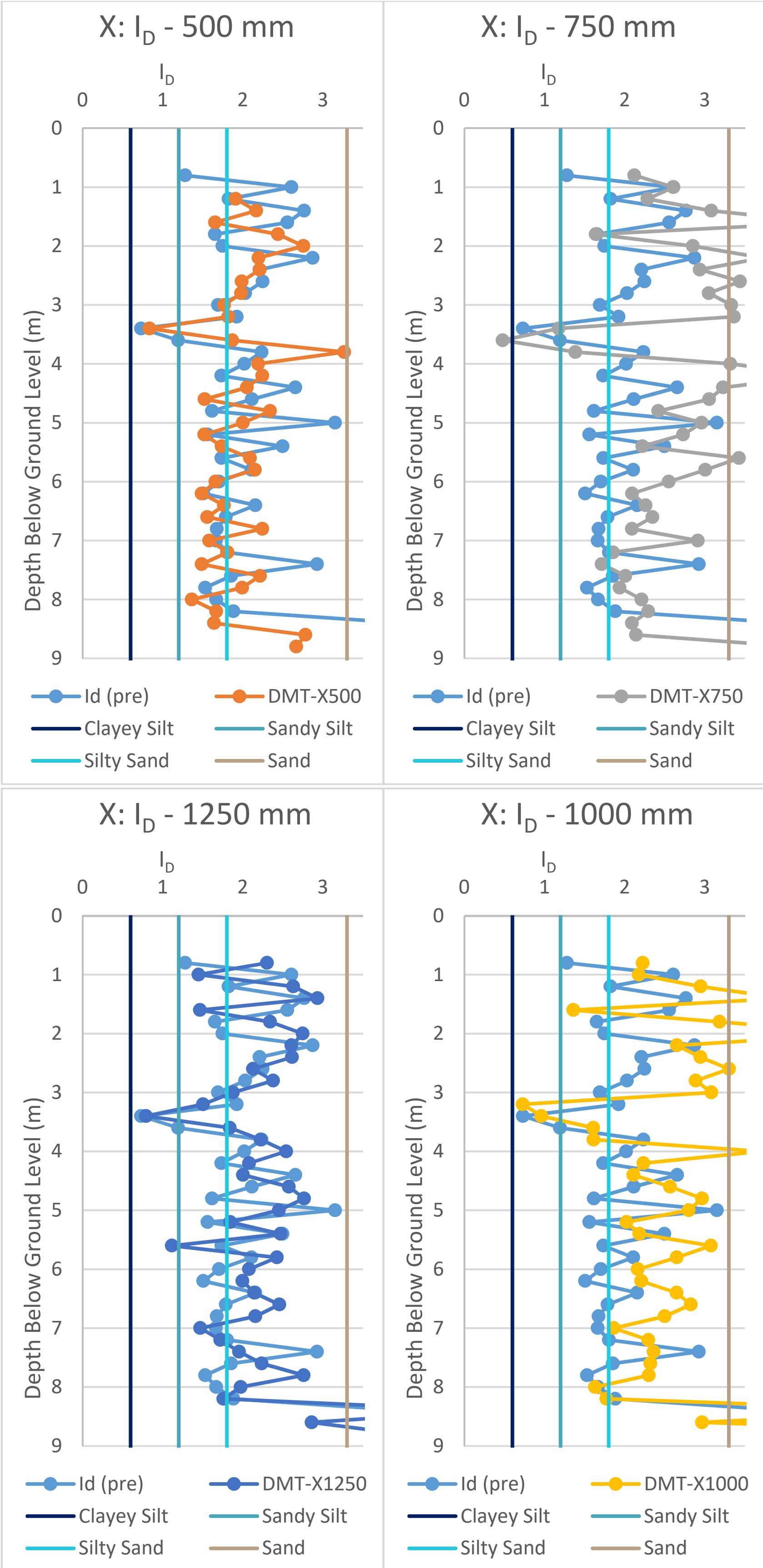
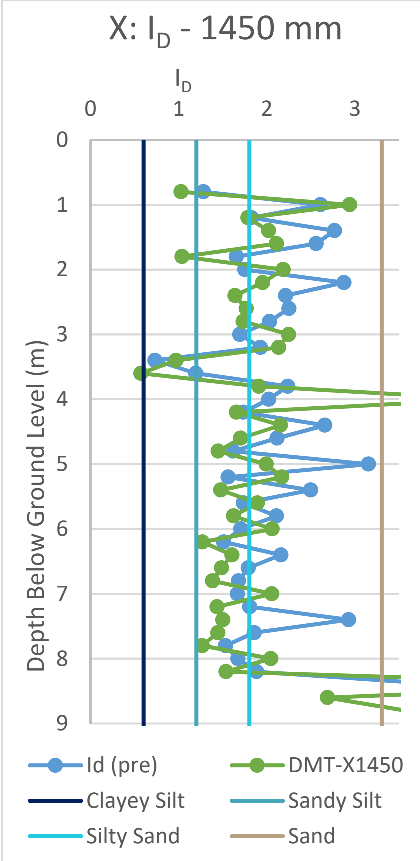
SEISMIC DILATOMETER TEST (S D M T)

Ground Investigation	Hiways Geotechnical	TEST
15-169	27 Shirley Rd	**VS Sec Post**
		15 JAN 2016

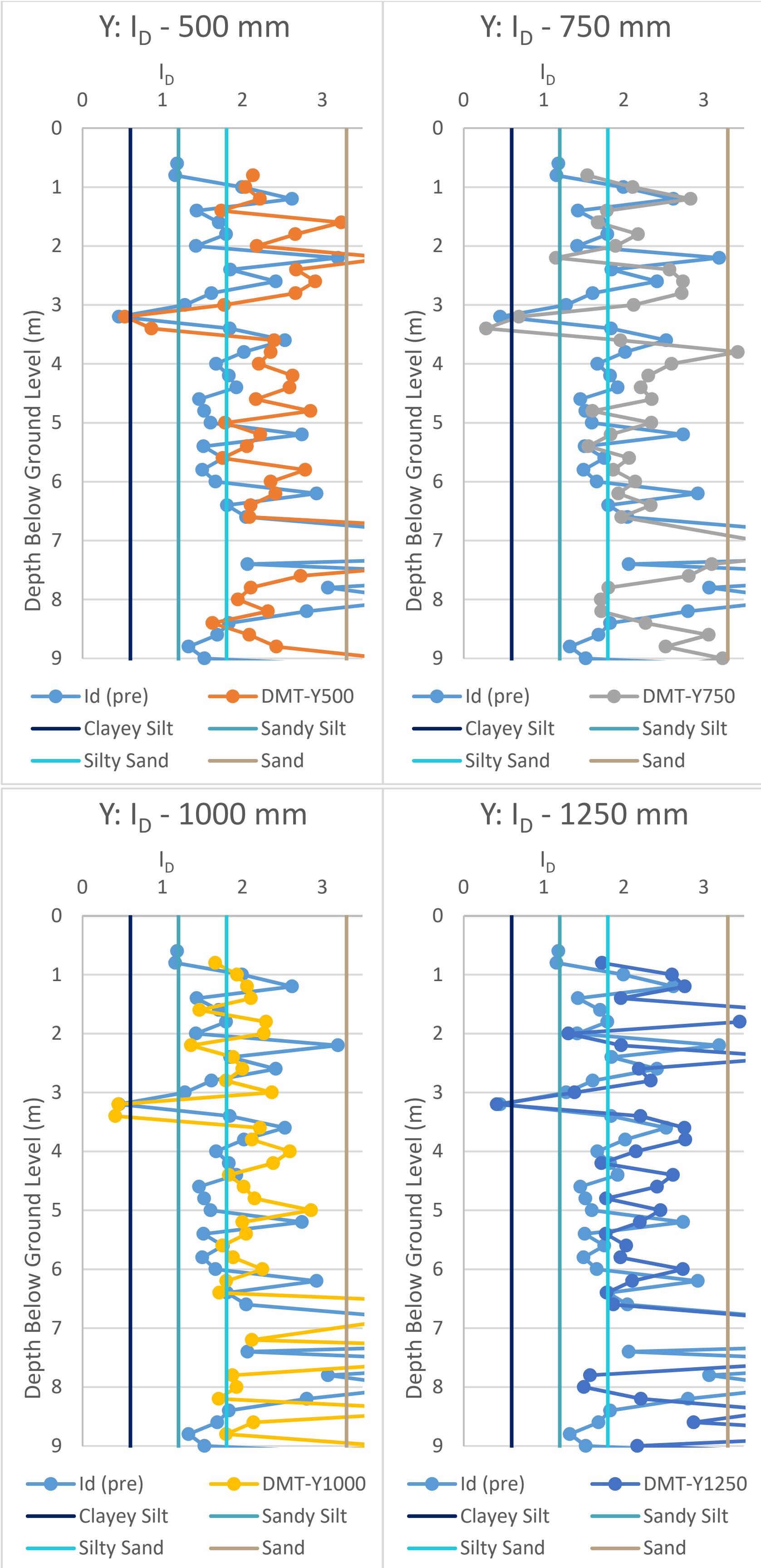


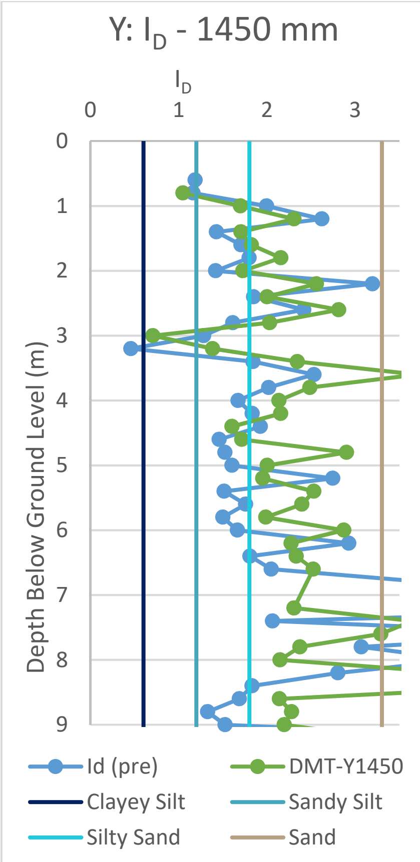


X: I_D - 500 mm
X: I_D - 750 mm
X: I_D - 1250 mm
X: I_D - 1000 mm
I_D
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Id (pre)
DMT-X500
DMT-X750
DMT-X1250
DMT-X1000
Clayey Silt
Sandy Silt
Silty Sand
Sand

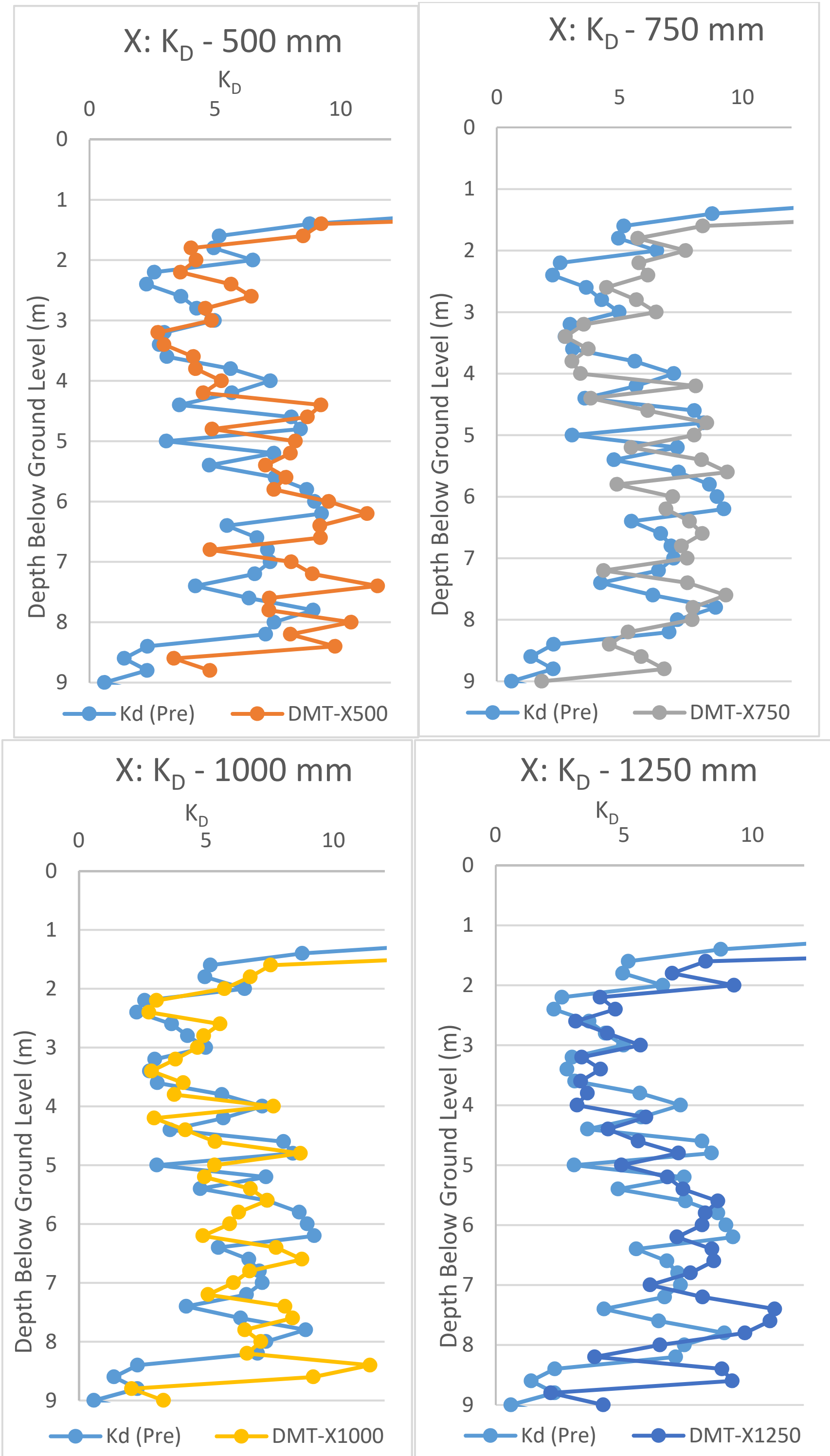


X: I_D - 1450 mm
I_D
0
1
2
3
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Id (pre)
DMT-X1450
Clayey Silt
Sandy Silt
Silty Sand
Sand

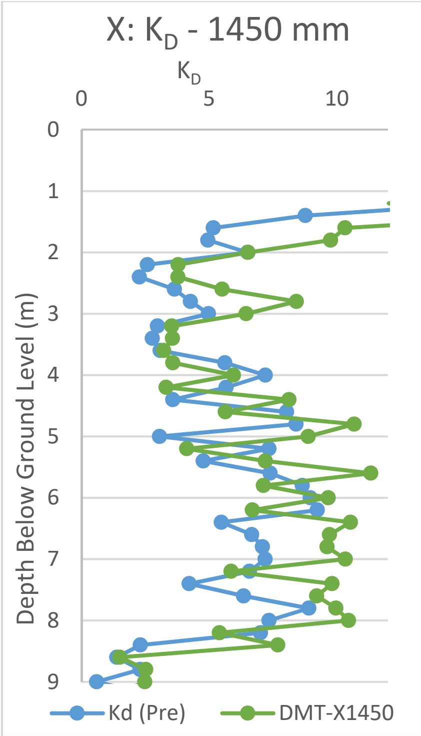

Y: I_D - 500 mm
Y: I_D - 750 mm
Y: I_D - 1000 mm
Y: I_D - 1250 mm
I_D
Depth Below Ground Level (m)
Id (pre)
DMT-Y500
DMT-Y750
DMT-Y1000
DMT-Y1250
Clayey Silt
Sandy Silt
Silty Sand
Sand

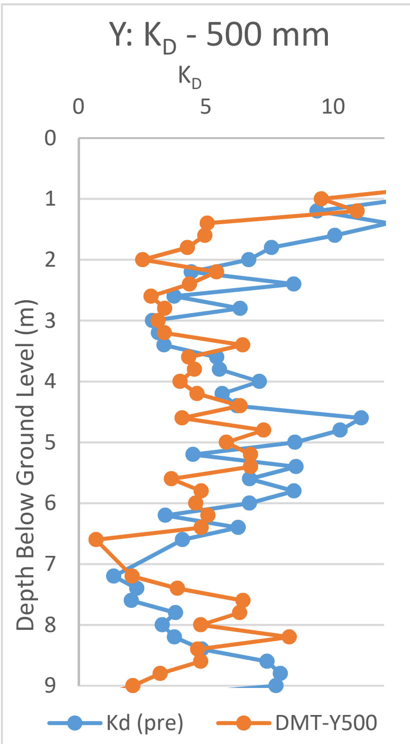


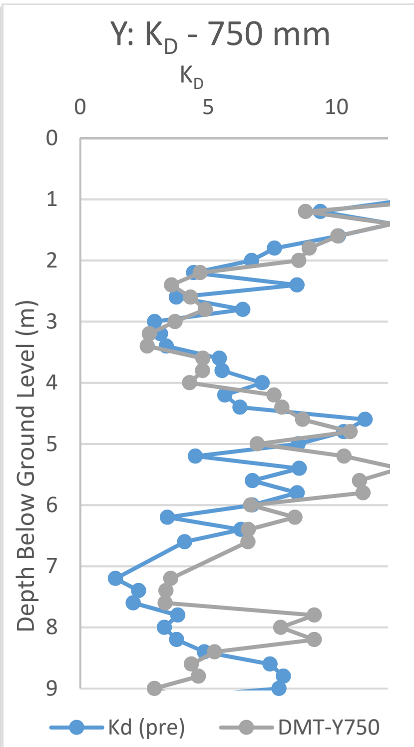
Y: I_D - 1450 mm
I_D
0
1
2
3
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Id (pre)
DMT-Y1450
Clayey Silt
Sandy Silt
Silty Sand
Sand

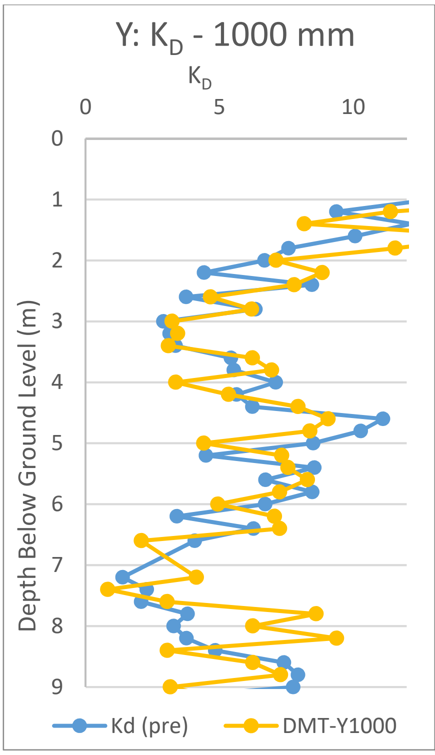


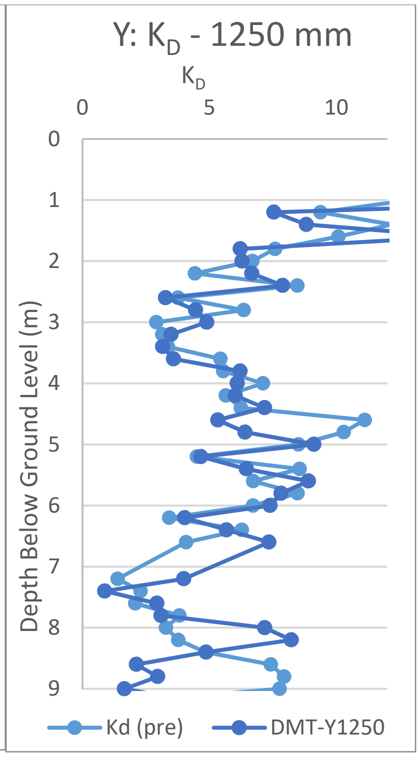
X: K_D - 500 mm
K_D
0
5
10
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Kd (Pre)
DMT-X500
X: K_D - 750 mm
0
5
10
Depth Below Ground Level (m)
Kd (Pre)
DMT-X750
X: K_D - 1000 mm
K_D
0
5
10
Depth Below Ground Level (m)
Kd (Pre)
DMT-X1000
X: K_D - 1250 mm
K_D
0
5
10
Depth Below Ground Level (m)
Kd (Pre)
DMT-X1250

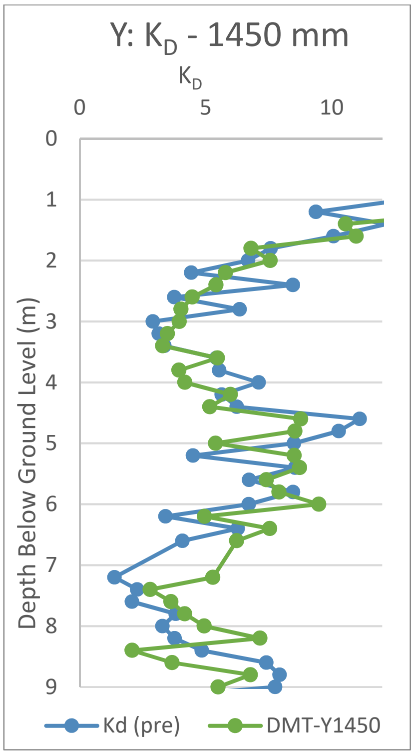

X: K_D - 1450 mm
K_D
0
5
10
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Kd (Pre)
DMT-X1450

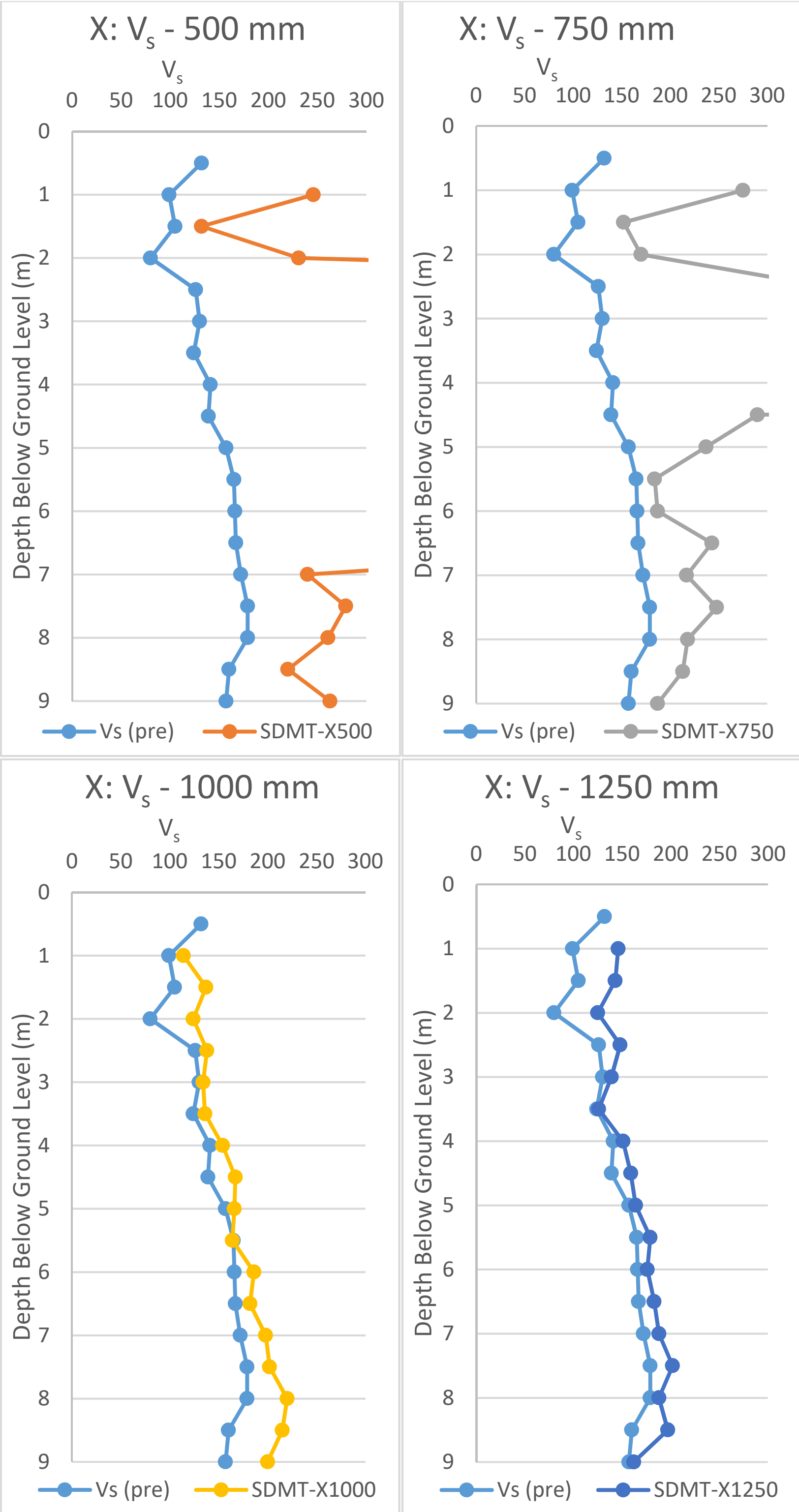

Y: K_D - 500 mm
K_D
0
5
10
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Kd (pre)
DMT-Y500

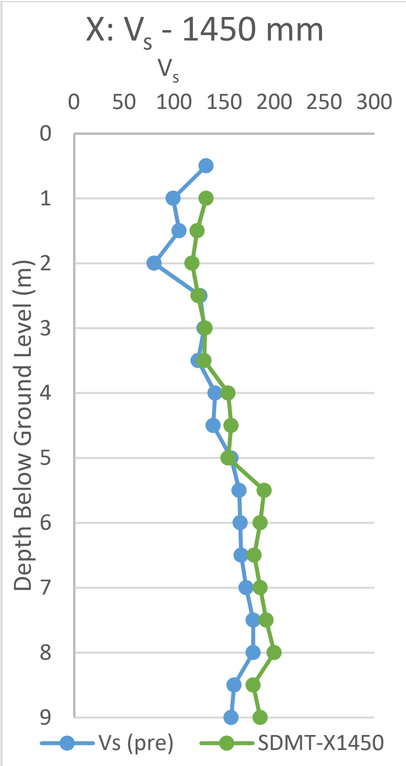

Y: K_D - 750 mm
K_D
0
5
10
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Kd (pre)
DMT-Y750

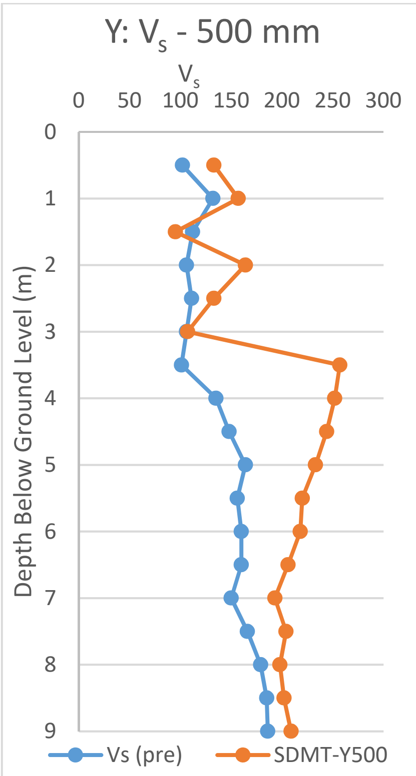

Y: K_D - 1000 mm
K_D
0
5
10
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Kd (pre)
DMT-Y1000

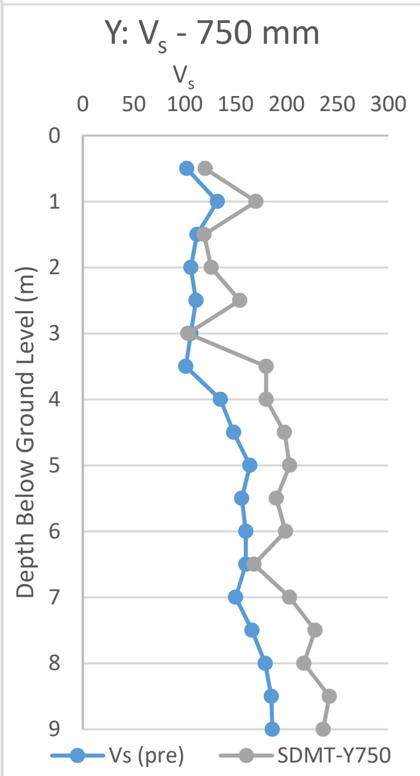

Y: K_D - 1250 mm
K_D
0
5
10
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Kd (pre)
DMT-Y1250

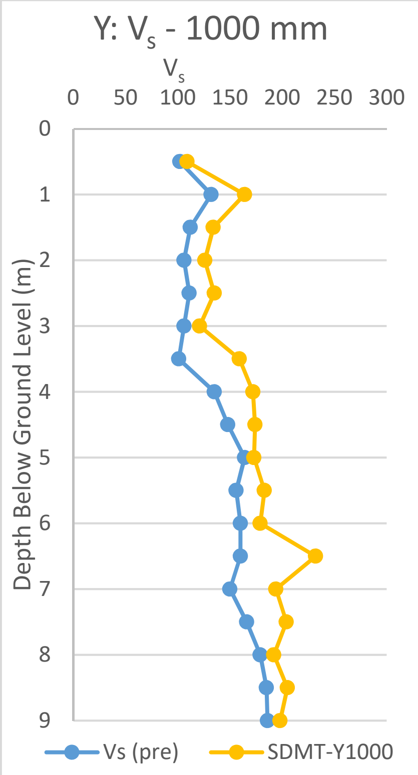

Y: K_D - 1450 mm
K_D
0
5
10
0
1
2
3
4
5
6
7
8
9
Depth Below Ground Level (m)
Kd (pre)
DMT-Y1450

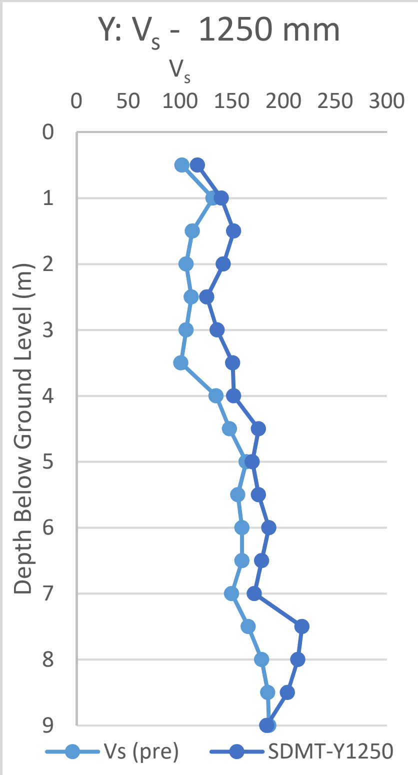

X: V_s - 500 mm
V_s
Depth Below Ground Level (m)
Vs (pre)
SDMT-X500
X: V_s - 750 mm
V_s
Depth Below Ground Level (m)
Vs (pre)
SDMT-X750
X: V_s - 1000 mm
V_s
Depth Below Ground Level (m)
Vs (pre)
SDMT-X1000
X: V_s - 1250 mm
V_s
Depth Below Ground Level (m)
Vs (pre)
SDMT-X1250

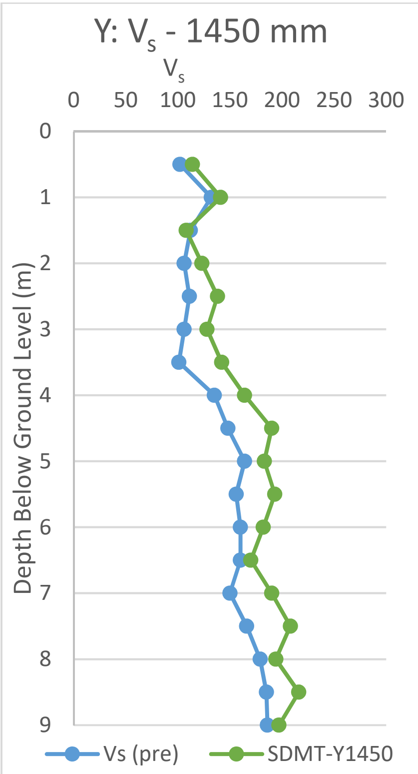

X: V_s - 1450 mm
V_s
0 50 100 150 200 250 300
0 1 2 3 4 5 6 7 8 9
Depth Below Ground Level (m)
Vs (pre)
SDMT-X1450

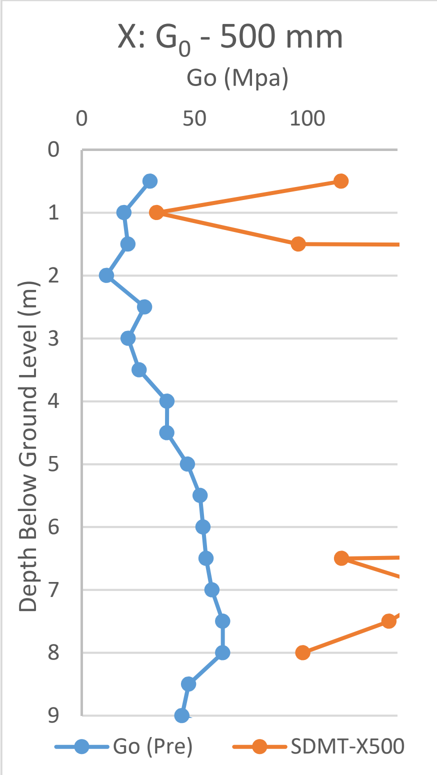

Y: V_s - 500 mm
V_s
0 50 100 150 200 250 300
0 1 2 3 4 5 6 7 8 9
Depth Below Ground Level (m)
Vs (pre)
SDMT-Y500

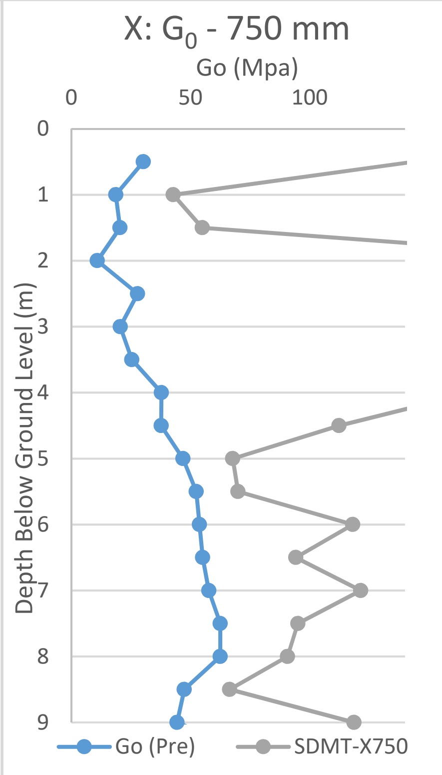

Y: V_s - 750 mm
V_s
0 50 100 150 200 250 300
0 1 2 3 4 5 6 7 8 9
Depth Below Ground Level (m)
Vs (pre)
SDMT-Y750

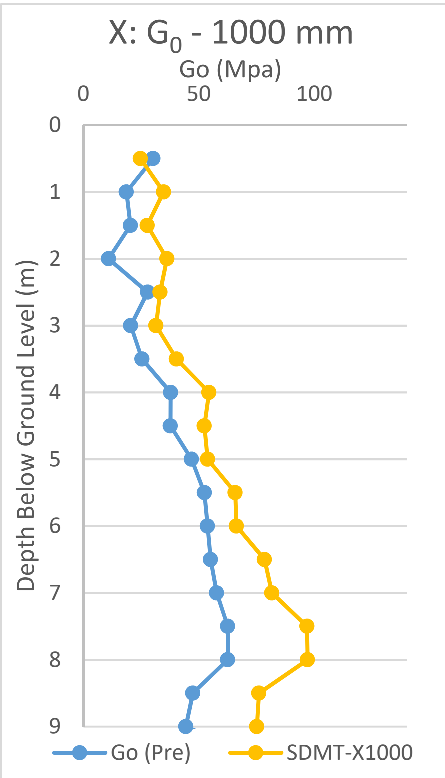

Y: V_s - 1000 mm
V_s
0 50 100 150 200 250 300
Depth Below Ground Level (m)
0 1 2 3 4 5 6 7 8 9
Vs (pre)
SDMT-Y1000

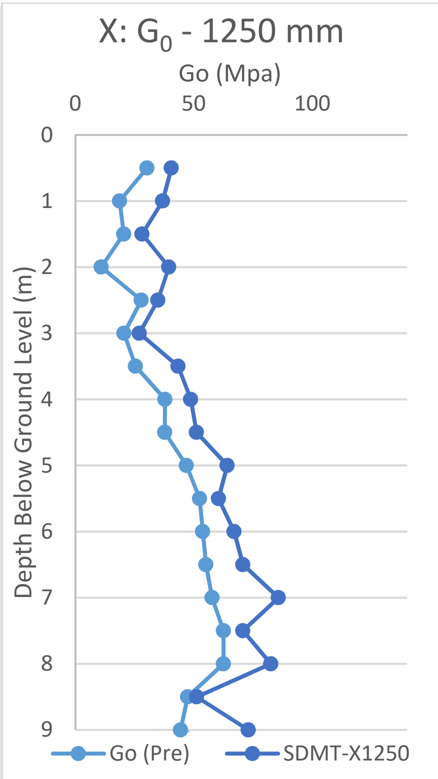

Y: V_s - 1250 mm
V_s
0 50 100 150 200 250 300
Depth Below Ground Level (m)
0 1 2 3 4 5 6 7 8 9
Vs (pre)
SDMT-Y1250

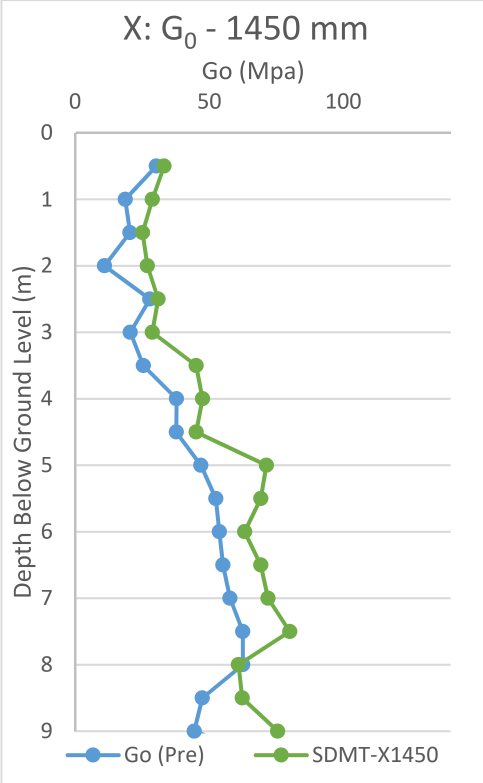

Y: V_s - 1450 mm
V_s
0 50 100 150 200 250 300
Depth Below Ground Level (m)
0 1 2 3 4 5 6 7 8 9
Vs (pre)
SDMT-Y1450

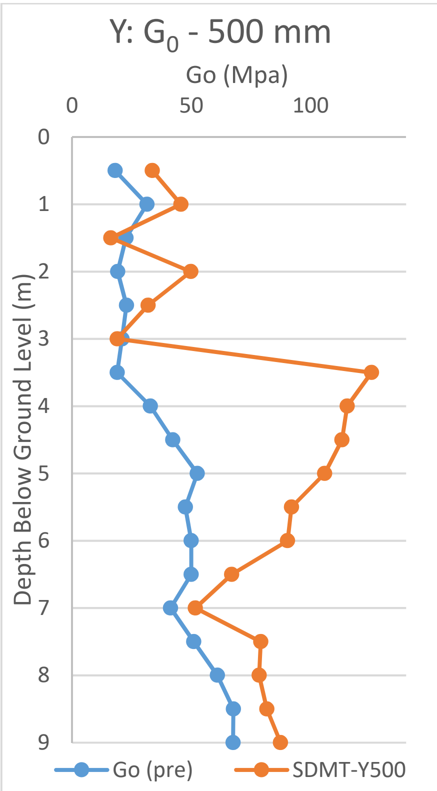

X: G_0 - 500 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (Pre)
SDMT-X500

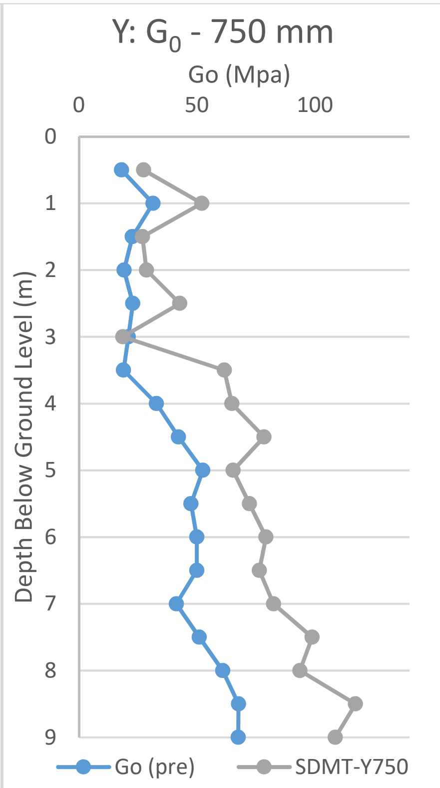

X: G_0 - 750 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (Pre)
SDMT-X750

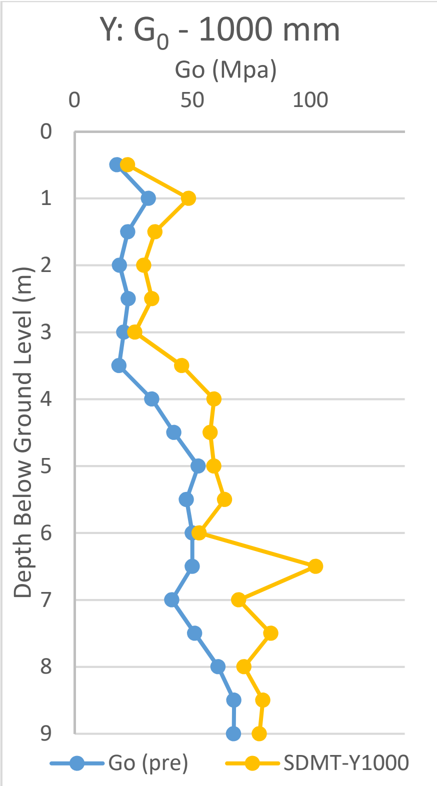

X: G_0 - 1000 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (Pre)
SDMT-X1000


X: G_0 - 1250 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (Pre)
SDMT-X1250

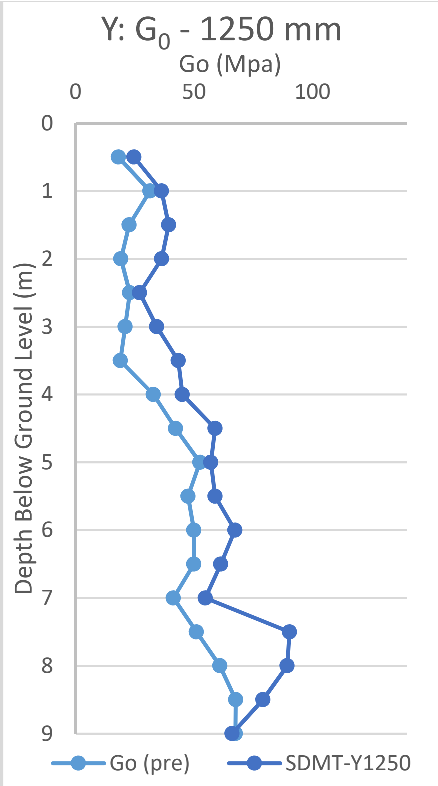

X: G_0 - 1450 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (Pre)
SDMT-X1450


Y: G_0 - 500 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (pre)
SDMT-Y500

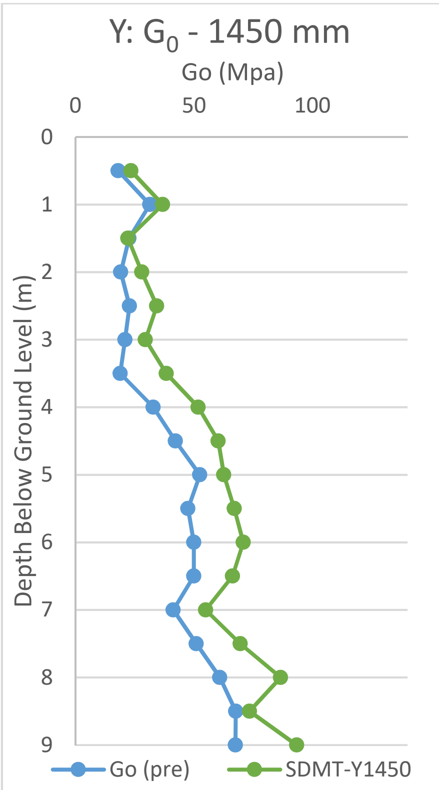

Y: G_0 - 750 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (pre)
SDMT-Y750



Y: G_0 - 1000 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (pre)
SDMT-Y1000

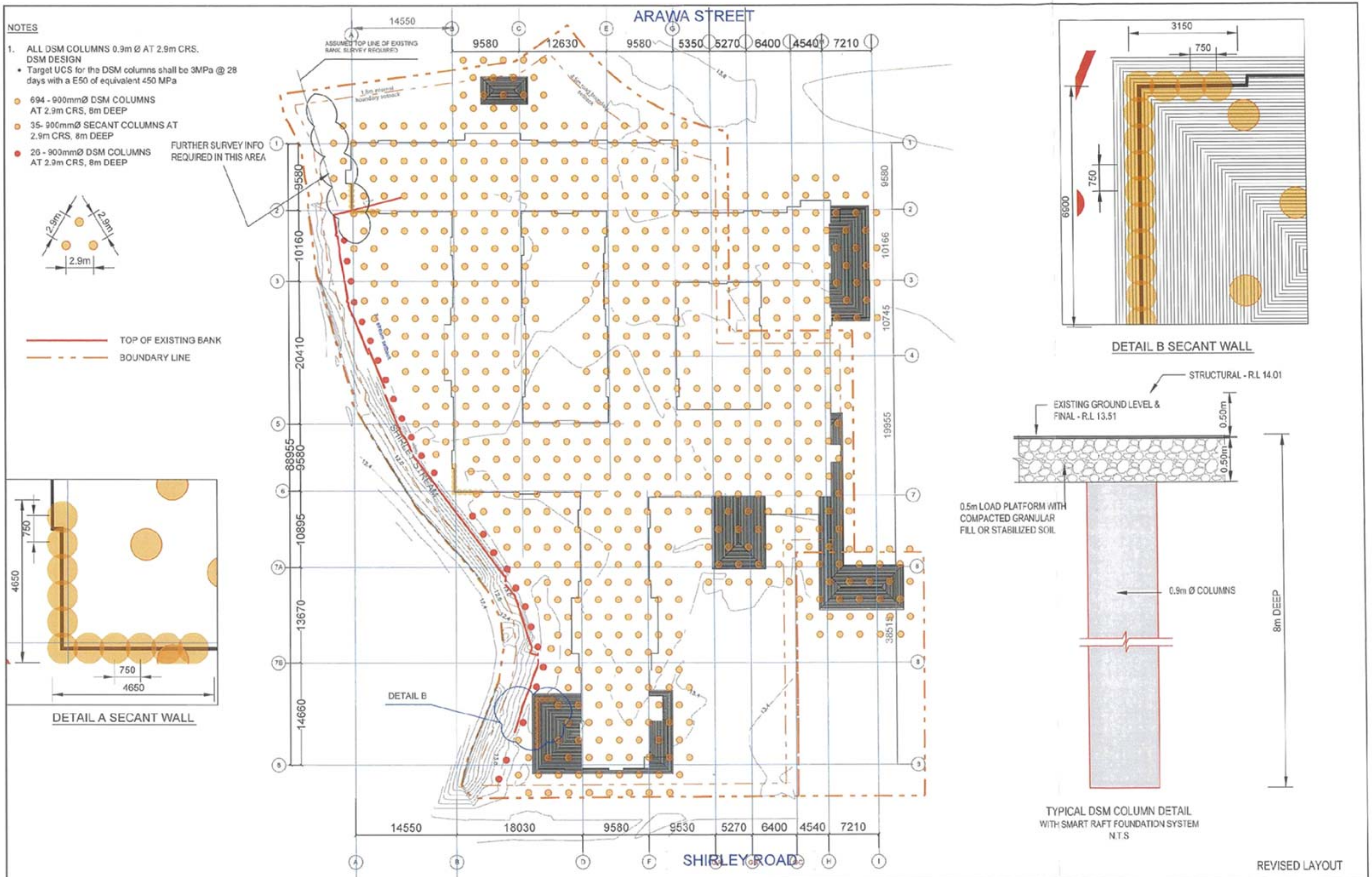


Y: G_0 - 1250 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (pre)
SDMT-Y1250



Y: G_0 - 1450 mm
Go (Mpa)
0
50
100
Depth Below Ground Level (m)
0
1
2
3
4
5
6
7
8
9
Go (pre)
SDMT-Y1450

Appendix M – Research Site DSM Plan



REV I COLUMNS CHANGED TO 8m DEEP AT 2.9m CENTERS, 15.8.15

Hiway GeoTechnical

WILLOWLEA RETIREMENT HOME
PROPOSED GROUND STABILIZATION

		DATE
DESIGNED	AOS	21/7/14
DESIGN CHECK	RW	19/3/15
DRAWN	KW	15/8/15
DWG CHECK	RW	15/8/15

SCALE 1:500
DRAWING No. 60297/1
SHEET 1 OF 1
CONTRACT No. –
REVISION I
A3 ORIGINAL

C:\Documents and Settings\mike\My Documents\hiways willowlea\60297-1 rev I -15.09.2015.dwg, 9/15/2015 2:27:22 p.m., DWG To PDF.pc3